



RIDGE

**ST JAMES'S HOUSE
STRUCTURAL ROBUSTNESS
ASSESSMENT REPORT
BRIGHTON AND HOVE CITY COUNCIL**
October 2023



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Prepared for

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1. INTRODUCTION

1.1. Site Address

St James's House
High St, Kemptown
Brighton and Hove
BN2 1RW

1.2. Structural Engineering Brief

Ridge and Partners LLP (Ridge) were appointed by Brighton and Hove city council to carry out structural investigations to determine the robustness of the dwelling block, St James's House, Kemptown, Brighton. The appointment came following owners of LPS dwelling blocks, including Brighton and Hove city council, being advised to seek professional advice regarding the safety of their assets by the Ministry of Housing, Communities & Local Government (MHCLG).

The brief was therefore to carry out an audit on the construction of the block, based on available historic information, followed by detailed intrusive investigations into selected areas of the block. The construction details of the block obtained from this audit would then form the basis of the structural assessment to determine whether the construction of the block was sufficient to resist progressive collapse in the event of accidental loading from an internal gas explosion.

1.3. Report Contents

The contents of this report relate exclusively to the construction of St James House and its structural condition at the time of inspection. The report has been compiled following the visual inspection and a series of intrusive and non-intrusive tests conducted on a limited number of pre-selected areas of the structure.

This report documents the main findings of the investigation and the findings of the subsequent structural assessment into the robustness of the block against disproportionate collapse.

1.4. Limitations

Throughout the duration of the intrusive investigations the block remained inhabited by residents. This presented challenges to the investigation team in terms of availability of vacant flats within which intrusive investigations could be undertaken. Three suitable flats were identified, although none were available at top floor level and as such no information was uncovered with regard to the connections at roof level.

Whilst the investigative works were detailed, with multiple tests carried out in each of the three flats, it should be noted that many areas of the block were not tested and thus the assessment of the block can only be based on what was uncovered in the sample investigation. The investigations were also only carried out from within the flats. No works were carried out externally or in the communal areas due to H&S concerns for the residents.

All flats within the block are single level (no duplex apartments). It was therefore not possible to obtain core samples from floor slabs within the flats during the investigative phase. Ridge advised the client that core sampling could be undertaken within a cupboard off the communal stairwell which would reduce the impact on residents from this intrusive works. However, this area had not been R&D tested, or asbestos stripped in time to allow the testing within these areas prior to authoring this report. The core samples will be obtained in a follow-up visit. The calculation for the key element checks on the floor slabs have therefore calculated the

required compressive strength to pass the assessment, which will be confirmed following the receipt of the lab test results.

1.5. Statement

The purpose of the Report is to advise on the construction of the LPS structure and its susceptibility to disproportionate collapse, together with those related matters specifically referred to therein and it is not intended to be used for any other purposes. The Report is for the sole benefit and may only be relied upon by the addressee, to whom we will owe a duty of care. The Report or any part of it is confidential to the addressee and should not be disclosed to any third party for any purpose, without our prior written consent of Ridge and Partners LLP as to the form and context of such disclosure. The granting of such consent shall not entitle the third party to place reliance on the Report, nor shall it confer any third-party rights pursuant to the Contracts (Rights of Third Parties) Act. The Report may not be assigned to any third party.

2. BUILDING CONSTRUCTION

St James's Houses is a 16-storey residential block with a lower ground car park. The construction of the property is estimated to have taken place circa 1965 and are of a Wates (LPS) design. . The car park at the lower ground level and the load-bearing structure of the ground floor primarily consists of in-situ reinforced concrete columns and beams, supporting precast floor panels at the first floor. This marks the initiation of the main construction using pre-cast wall and floor panels, which continue upwards. The lower ground floor car park is enclosed by in-situ reinforced concrete walls that provide support to the adjacent roads. Internal dividing walls at ground level and the elevator shafts also seem to be constructed using precast concrete units. The cavity walls at the ground floor level are likely to be non load bearing.

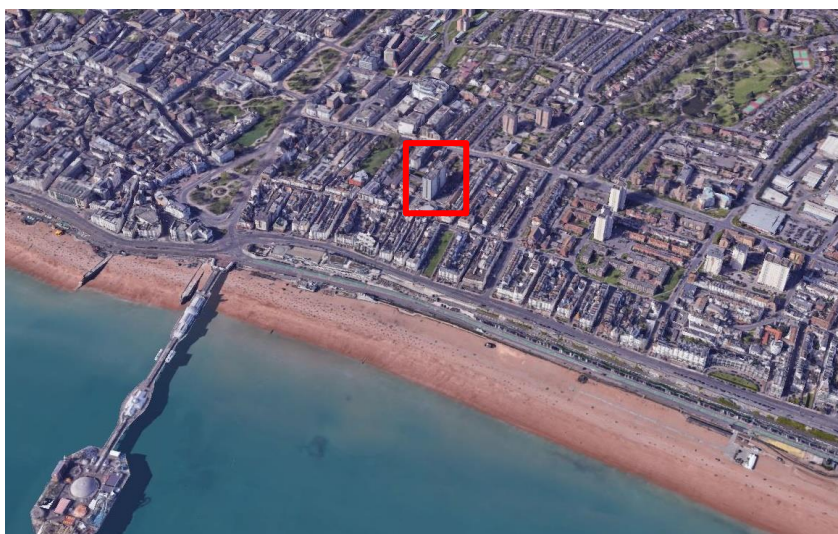


Figure 1 – St James House Location (Google Earth, 2023)



Figure 2 – St James's House (Google Maps, 2023)



Figure 3 - St James's House (Google maps, 2023)

3. EXECUTIVE SUMMARY

The Large Panel System (LPS) dwelling block, St James's House, Kemptown, Brighton has been assessed for its robustness to resist accidental loading and its susceptibility to progressive collapse.

A select number of flats were subjected to intrusive and non-intrusive investigative methods, including visual inspection, concrete testing and intrusive opening-up works. The results of the investigations were documented and used as the basis of the structural assessment.

The assessment was carried out in accordance with BRE Report 511. The document states that LPS blocks can be assessed under three criteria, of which a block needs only pass one. The criteria and results relating to St James's House are as follows:

LPS Criterion 1 – Adequate Ties (Reinforcement) within Joints

ST JAMES'S HOUSE – ADEQUATE TIES	
JOINT	PASS / FAIL
Flank Wall	Insufficient *
Cross Wall	Insufficient
Walls to Wall Joints	Insufficient
CONCLUSION - FAIL	

* The Flank Wall joints appear to have additional bars stitched in, potentially as a remedial measure. However, regardless, the block will not pass as the other joint types fail.

LPS Criterion 2 – Adequate Strength to Resist Accidental Loads

To satisfy LPS Criterion 2 each of the main structural members are subjected to 'key element' checks to determine whether they have sufficient strength to withstand the loading from accidental overpressures caused by an internal gas explosion. The magnitude of the overpressure to be tested is determined on the three criteria below:

- A. An LPS dwelling block with a piped gas supply within or to any part of the building: an assessment overpressure of 34 kN/m² should be used generally throughout the building.
- B. An LPS dwelling block with a basement: an assessment overpressure of 34 kN/m² should be used in the basement and in any other zone where an explosive mixture of gas might accumulate (potentially from an external source).
- C. An LPS dwelling block without a basement and without a piped gas supply to any part of the building: an assessment overpressure of 17 kN/m² should be used [to allow for explosions from sources such as bottled gas, large aerosols and cannisters- brought into the block by residents / others]

St James's House has a communal heating system served by a gas boiler within the basement of the block. The basement is formed by an insitu concrete frame (not LPS). There is no gas supply to the upper floors which are constructed from the LPS form of construction. We would therefore consider that the LPS sections of St James's House be assessed against the lower 17kN/m² overpressure.

Each of the LPS structural elements forming the block have been subjected to key element checks, with the results presented in the following table:

ST JAMES'S HOUSE – KEY ELEMENT CHECKS (WITHOUT PIPED-GAS SUPPLY – 17KN/M ²)	
JOINT	PASS / FAIL
Flank Wall	Inadequate
Cross Wall	Inadequate
Floor Slabs	Inadequate
CONCLUSION - FAIL	

LPS Criterion 3 – Ability to Mobilise Alternative Load Paths

Due to the number of structural elements that fail in the event of an explosion, it is unlikely the block would be able to develop adequate load paths to prevent disproportionate collapse in its current state.

ST JAMES'S HOUSE -16-STOREY BLOCK
Unable to mobilise alternative load paths
CONCLUSION - FAIL

The conclusion St James's House fails all the LPS assessment criterion and is therefore considered to be inadequately robust to resist disproportionate collapse.

Recommendations

To address the failings of the disproportionate collapse requirements, works are required to the block. We would therefore recommend that the required remaining life of the block should be discussed. It is likely that, if the blocks are to be retained long-term, that this will include strengthening works.

As a short-term, risk-reduction measure the piped-gas supply in the lower ground floor should be removed to mitigate any risk for disproportional collapse. This process should be commenced as soon as practicably possible.

A risk analysis has been carried out to determine:

- Whether the risk can be reduced to an acceptable level through risk-reduction measures for the duration of the remaining life of the blocks.
- Whether risk-reduction measures are not alone sufficient, and strengthening works are required.

Along with cost-benefit analysis, accounting for the short remaining life of the blocks, to understand whether the strengthening works are suitable.

Undertake an analysis of the building in the fire condition with a suitably qualified Fire Engineer and Structural Engineer to assess its ability to resist fire given the low covers experienced during the robustness assessment.

Accelerated demolition programme may need to be considered following the results of the cost-benefit analysis. Further information regarding the recommendations can be found in Section 8.

4. BRIEF HISTORY OF LPS BLOCKS AND DISPROPORTIONATE COLLAPSE

On the 11th March 1968 construction was completed on a 21-storey dwelling block in Newham, East London, called Ronan Point. Two months after opening, the block of flats suffered progressive collapse to the south-east corner of the structure. A subsequent Tribunal found that the partial collapse was caused by an explosion of town gas in one of the flats. The explosion had caused the loadbearing flank wall of the flat to 'blow out', thus removing the support to the other loadbearing elements and causing further elements to fail. This event sparked a series of changes to legislation related to the design of new LPS structures and required the existing LPS building stock to be assessed.

Since this date several further documents have been produced to provide advice on the structural assessment of LPS buildings from leading professional bodies such as the Institution of Structural Engineers and the Building Research Establishment (BRE). Today the assessment process of LPS blocks is generally carried out based on the guidance provided in the BRE Report 511 – Handbook for the Structural Assessment of Large Panel System (LPS) Dwelling Blocks for Accidental Loads.

In more recent times, an investigation undertaken on the Ledbury Estate in 2017 showed that the LPS blocks were insufficiently robust to resist disproportionate collapse. Subsequent to this, the government wrote to local councils who owned LPS blocks within their housing stock to request that they be subjected to structural assessment.

5. ST JAMES'S HOUSE INVESTIGATION

5.1. Investigation Overview

The dwelling block, St James's House located in Kempton, Brighton has been assessed for its robustness to resist accidental loading from over-pressure, such as an internal gas explosion, and its susceptibility to progressive collapse. The residential block was designed by Wates and the Brighton Borough Architects department. The consulting engineers were Arup & Partners. It is likely to be constructed circa 1965.

5.2. Investigation Methodology

In the absence of the construction details of the blocks Ridge subjected three selected flats for both intrusive and non-intrusive investigation works to determine its construction, including:

- Visual Inspection
- Concrete Reinforcement Scanning
- Concrete Testing (insitu & laboratory)
- Intrusive Opening Up Works

5.3. Main Findings of the Investigation

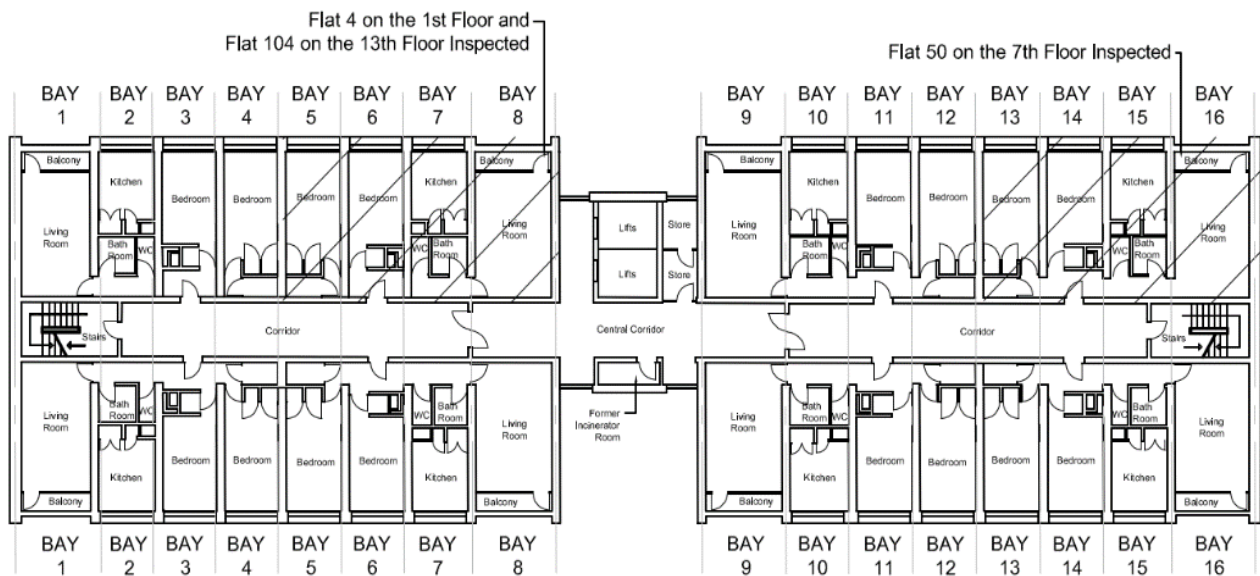


Figure 4 – Typical floor plan for St James's House

Floor Slab Construction

Span: 3.5m (max.)

Construction: 150mm thick, solid concrete floor slab

Bottom Reinforcement: 10mm ribbed bars, 250mm c/c + 6mm diameter U-bars @ slab edge

Top Reinforcement: Unreinforced

Flank Wall Construction

Height: 2.4m

Construction: ~180mm thick concrete loadbearing wall panel

Reinforcement: Internal face: 4no 12mm diameter dowel bars @ 340mm c/c + 1no intermediate 24mm square twisted bar between dowels

External face: Unknown – too deep to scan but believed unreinforced

Cross Wall Construction

Height: 2.4m

Construction: 180mm thick concrete loadbearing wall panel

Reinforcement: 5mm round bars, 150mm c/c

Spine Wall Construction

As per Cross Wall

Joint Construction

The following annotated photographic logs show the various joint details between the load bearing members.

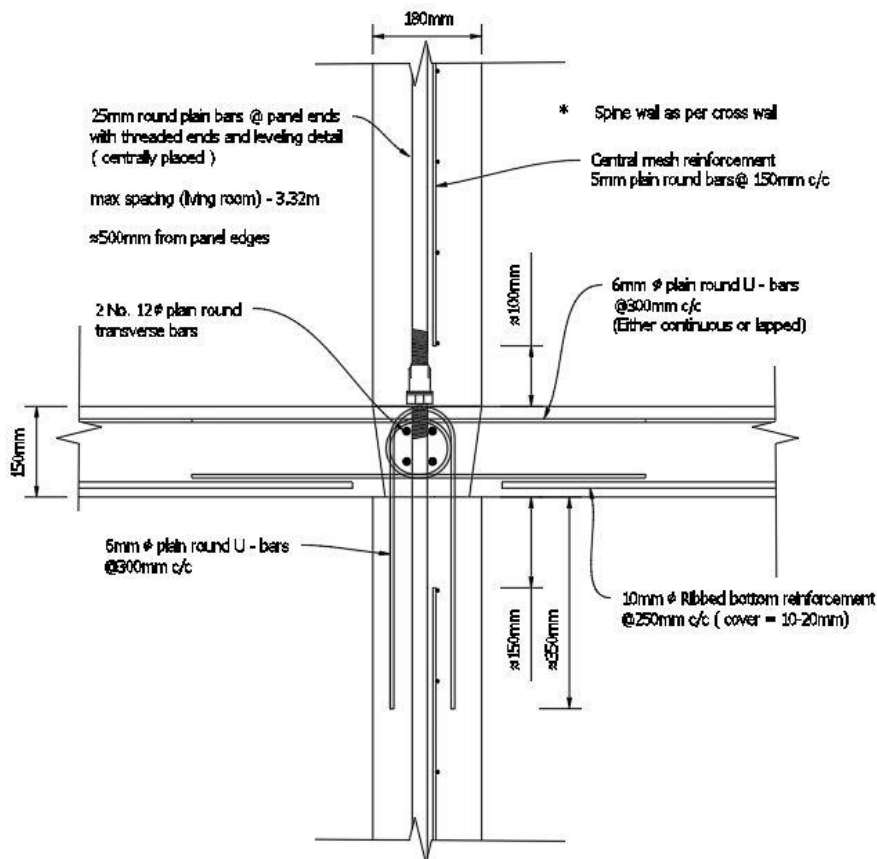


Figure 5 – Cross wall / floor slab detail

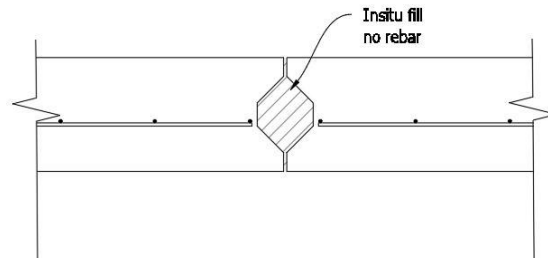


Figure 6 – Cross wall vertical joint

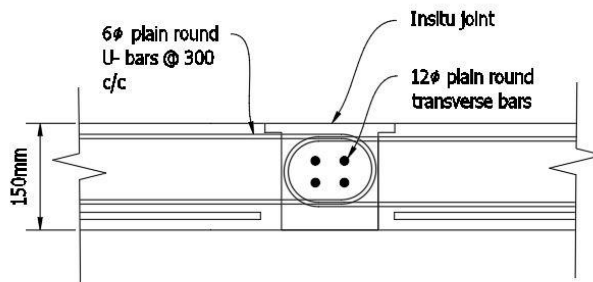


Figure 7 – Floor to floor joint in hallway (no cross wall present)

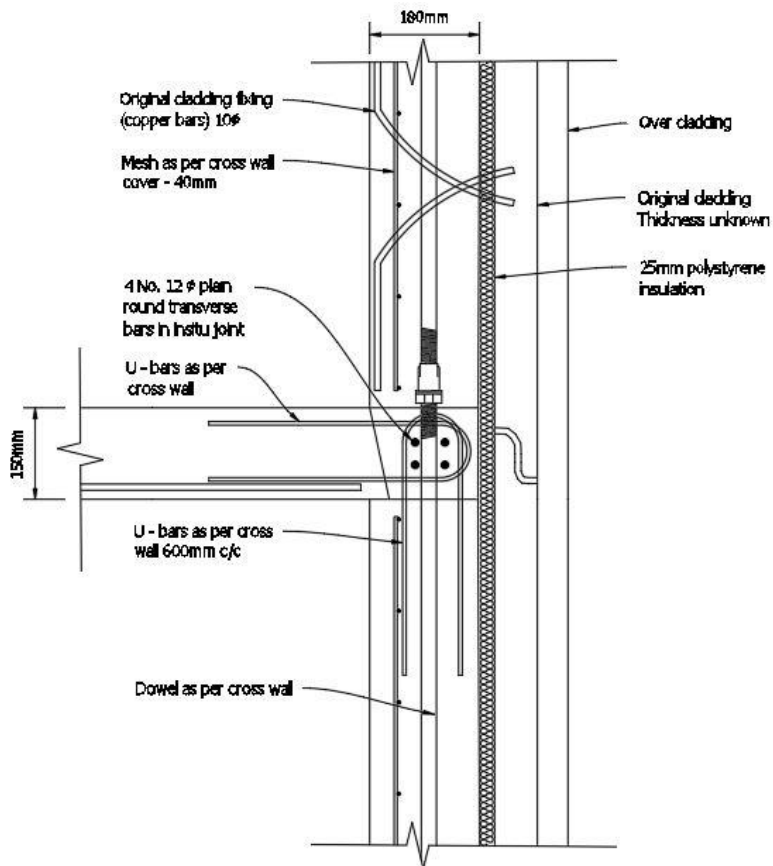


Figure 8 – Flank wall

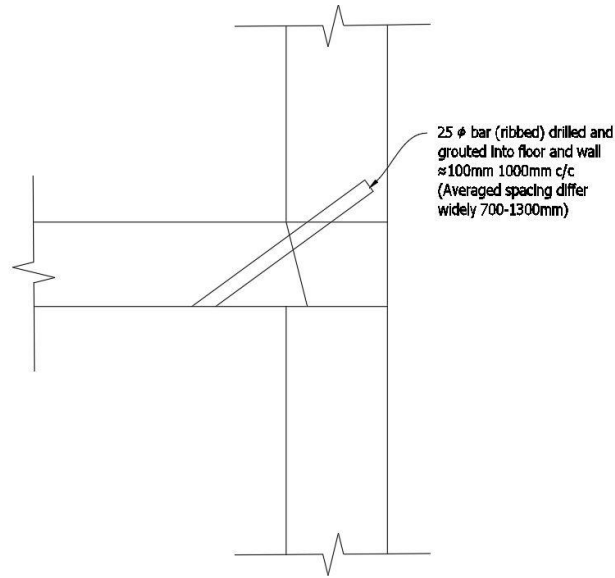


Figure 9 – Remedial ties for flank and spine walls (section)

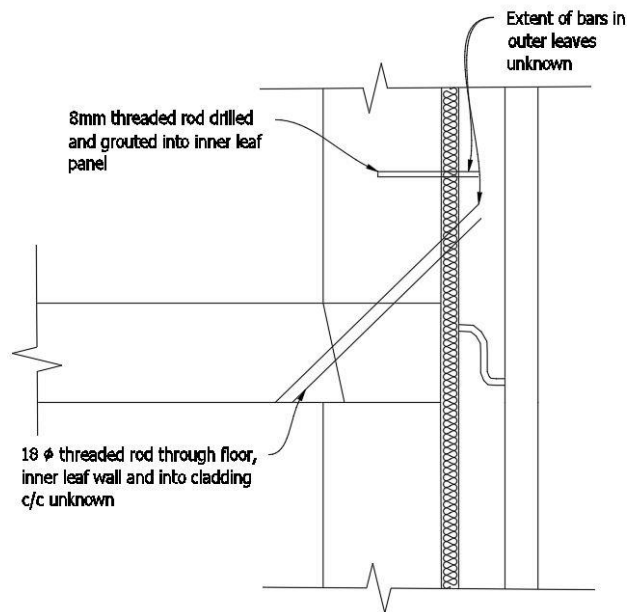


Figure 10 – Cladding ties flank wall section

5.4. Concrete Testing

Carbonation

Carbonation testing is an intrusive, non-destructive testing method which determines the depth to which carbon dioxide in the atmosphere has penetrated the concrete. The cement paste in concrete generally has a pH of around 13 which creates a passive environment around the reinforcement, preventing corrosion. However, over time carbon dioxide diffuses into the concrete, which reduces the alkalinity of the concrete, subsequently losing passivity and its protection to the reinforcement within. Carbonation is not detrimental to the concrete until the passivity front has reached/exceeded the depth of the embedded steel. Once the passivity front has surpassed the reinforcement, and in the presence of moisture, the steel will begin to actively corrode and expand. This expansion creates internal pressure in the concrete and causes the concrete to crack and spall around the reinforcement. This test assesses the risk of corrosion to the reinforcement.

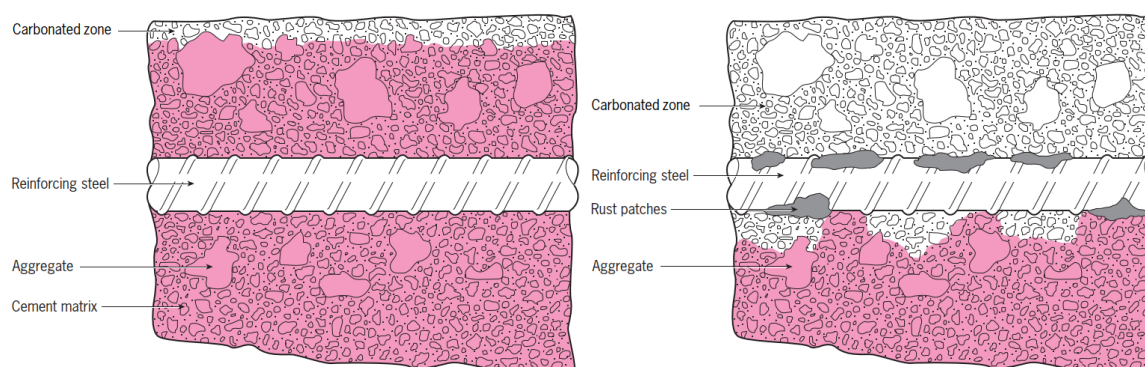


Figure 11 – (Left) Diagrammatic view of steel protected from carbonation-induced corrosion in partially carbonated concrete, (Right) Diagrammatic view of steel corroding in carbonated concrete. (BRE, 2000)

The testing was carried out by breaking out a small section of the concrete with a hammer drill. All the dust on the surface of the freshly exposed face was then removed with an air pump to prepare the surface for the testing. The indicator, phenolphthalein solution, was then applied to the freshly exposed surface using a pipette. The indicator turned pink when in contact with the concrete with a pH exceeding 9 and remained clear at a pH lower than 9. Concrete which turns pink is still providing a protective environment for the reinforcement, whereas the concrete which remains colourless has carbonated and would no longer be providing protection to any reinforcement which was located at this depth.

The results from the carbonation testing should only be used as a guide for the true depth of carbonated concrete. It has been suggested that the true passivity front extends between 5-10mm beyond the carbonation depth indicated using phenolphthalein solution. However, in areas which have high chloride content, this can be as much as 20mm beyond the indicated depth. These two limits should therefore be considered when assessing the risk of corrosion to the embedded reinforcement.

The carbonation depth was measured, from the face of the member to where the concrete turns pink, using a tape measure/callipers and recorded. The depth of carbonation recorded was then compared to the depth of the reinforcement to determine whether the passivity front had reached the reinforcement. Carbonation testing was carried out on all the anchor blocks which were safely accessible. The testing produced similar readings for the different test locations. The results of the carbonation tests are in Table 1 below.

CARBONATION TEST RESULTS					
Flat	Test Location	Member Type	Carbonation Depth (mm)	Min. Cover to Bar (mm)	Carbonation Surpassed Reinforcement?
Flat 16	1	Cross Wall 1	10	21mm to U bar 75mm main rebar	No No
Flat 16	2	Ceiling 1	25	10	Yes
Flat 16	3	Cross Wall 2	5	26mm to U bars 80-100mm main rebar	No No
Flat 16	4	Ceiling 2	10	10mm	Yes
Flat 78	5	Flank Wall	10	40mm	No
Flat 78	6	Cross Wall	10	80mm mesh 60mm U bars	No No
Flat 78	7	Ceiling 1	10	13mm main bars 20mm U bars	No No
Flat 78	8	Ceiling 2	10	15mm main bars 20mm U bars	No No
Flat 99	9	Flank Wall	5	58mm U bars 70mm mesh 12mm edge bar	No No No
Flat 99	10	Cross Wall	15	50mm mesh 15mm U bars	No Yes
Flat 99	11	Ceiling 1	15	4mm main bar 30 U bars	Yes No
Flat 99	12	Ceiling 2	10	6mm main bars 20mm U bars	Yes No

Table 1 - Carbonation Depths

In the majority of the twelve locations, the carbonation depth was observed to be relatively shallow and had not surpassed the depth of the embedded reinforcement, indicating that the rebar remains within a passive environment. However, the ceiling within flat 16 & flat 99 was shown to have a carbonation depth exceeding the reinforcement depth (due to the high chloride content, see Table 2) will also likely have carbonated to a depth exceeding the cover to the reinforcement.

Chlorides

Chloride testing was carried out by drilling the concrete with a hammer drill and the dust created collected and transferred into sealable bags. 12 no. dust samples were collected from across the three flats to be tested. The samples were then sent to Sandberg LLP's Clapham laboratory to conduct laboratory testing. The site is a UKAS accredited testing laboratory No. 0262.

Chlorides in concrete come from two sources. The first are cast-in chlorides which are present in the concrete mix at the time of casting typically from admixtures, some sources of aggregates and the cement. The second is ingressed chlorides which comes from airborne salt in the environment the concrete is exposed to. Chlorides within concrete can also take two forms; fixed chlorides (chemically/physically bound to the cement), or free (present in the pore water within the concrete).

It is the free chlorides that are responsible for the deterioration of the reinforcement. Free chlorides ingress through the concrete overtime towards the reinforcement. Once this has reached the reinforcement the free chlorides react with the protective oxide layer which forms around the reinforcement within the concrete and causes localised breakdown of this layer. This allows localised corrosion to initiate on the reinforcement.

The BRE have published a series of diagrams in Digest 444 part 2 which can be used as a part of the assessment of chloride levels in concrete members, for 25, 40 & 60 year old structures. The diagrams show the risk of reinforcement corrosion within concrete elements for the given conditions for the respective age groups. The building had been constructed circa 1965, meaning the property is roughly 58 years old at the time of inspection. The concrete testing results will therefore be compared against the BRE 444 diagram for a 60 year old structure, as this best represents the structure. This diagram is shown in Figure 12.

4c 60-year-old concrete structures (extrapolated data)

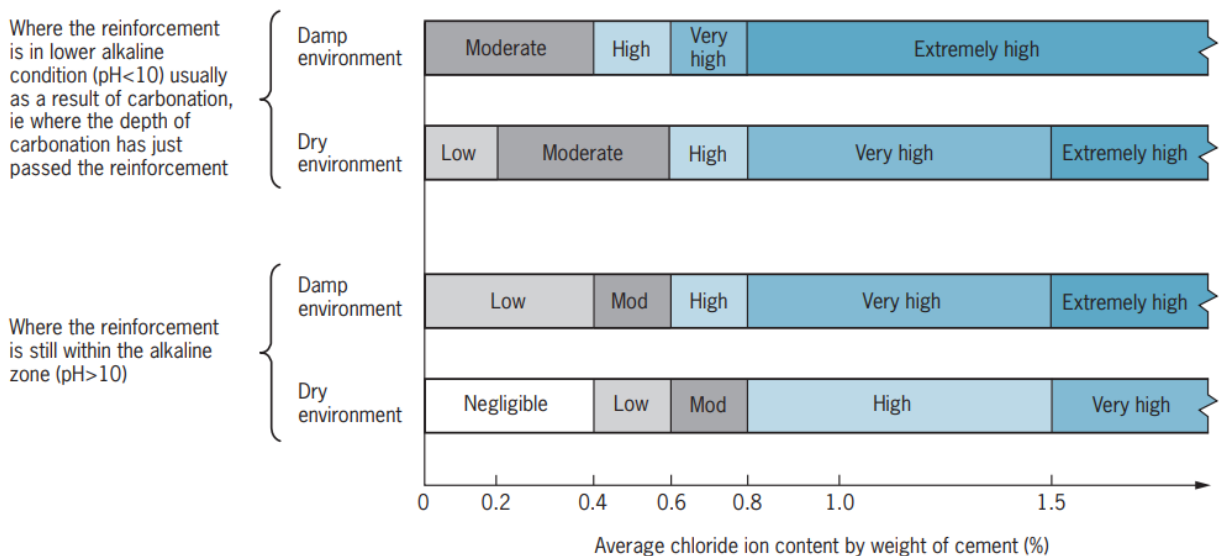


Figure 12 – Estimated Risk of Steel Reinforcement Corrosion Associated with Carbonation, Cast-in Chloride Content and Environment Conditions (BRE, 2000)

The testing data has been assessed based on the BRE guidance to created Table 2, showing the risk of steel reinforcement corrosion in each of the areas tested.

CHLORIDE TESTING						
	Test Location	Member Type	Carbonation Reached / Surpassed Reinforcement	Atmosphere	Chloride Cl % by Mass of Cement	Risk of Steel Reinforcement Corrosion (BRE Digest 444 pt1)

Flat 16	1	Ceiling 1	Yes	Dry	0.07	Low
Flat 16	2	Ceiling 2	Yes	Dry	0.07	Low
Flat 16	3	Cross Wall 1	No	Dry	0.03	Negligible
Flat 16	4	Cross Wall 2	No	Dry	0.10	Negligible
Flat 78	5	Flank Wall	No	Dry	0.37	Negligible
Flat 78	6	Cross Wall	No	Dry	0.07	Negligible
Flat 78	7	Ceiling 1	No	Dry	0.24	Negligible
Flat 78	8	Ceiling 2	No	Dry	0.07	Negligible
Flat 99	9	Cross Wall 1	Yes (only U bars)	Dry	0.48	Moderate
Flat 99	10	Flank Wall	No	Dry	0.33	Negligible
Flat 99	11	Ceiling 1	Yes	Dry	0.34	Moderate
Flat 99	12	Ceiling 2	Yes	Dry	0.17	Low

Table 2 - Interpretation of Sandberg Chloride Content Testing with BRE Digest 444 Part 1

Based on the results of the testing, compared using the above diagram, suggest the following:

- Most of the members were noted to have reinforcement within the un-carbonated zones and therefore offered protection from the alkalinity of the concrete. The increased risk of corrosion of some members tested is therefore related to the concentration of chlorides. The Cross Wall 1 in Flat 99 was found to be considerably high chloride content and is therefore also at high risk of corrosion to the embedded reinforcement, despite the depth of the reinforcement.
- The block is in a coastal area and the all the members that were tested were internal. It is likely this degree of chloride concentration has ingressed over the lifetime of the structure. It is also likely the chloride content in the concrete members may be attributed to admixtures in the concrete mix.
- Seven of the twelve internal concrete elements tested were found to be at negligible risk of corrosion to the embedded reinforcement.

However, it should be noted that the above statement refers to the embedded reinforcement within the precast concrete panels only – poor compaction of the insitu concrete / grout (the joints between panels) has created air pockets around the reinforcement. This has allowed the bars to actively corrode.

Cement Composition

The structural performance of concrete is greatly affected by the % content of cement, and the composition of the cement. Concrete with a low cement content, or incorrectly proportioned composition, may impact on the overall structural integrity of the structure and may provide a less protective environment to the reinforcement, leading to corrosion issues and subsequent spalling.

The results of the chemical analysis were then interpreted to understand the percentage weight of each chemical component against the total weight of the binder, shown in Table 4. This was then compared to the

requirements from BS EN 197-1:2011 – “Cement. Composition, specification and conformity criteria for common cements” as a guide to determine whether the cement composition would be acceptable to today's standards, shown in Table 5.

BS EN 197-1:2011, Section 5.2.1 states that “Portland cement clinker is a hydraulic material which shall consist of at least two-thirds by mass of calcium silicates ($3\text{CaO} \cdot \text{SiO}_2$ and $\text{CaO} \cdot \text{SiO}_2$), the remainder consisting of aluminium and iron containing clinker phases and other compounds. The ratio by mass (CaO) / (SiO_2) shall be not less than 2.0.” (BSI, 2011).

CEMENT COMPOSITION RESULTS				
% by weight of sample (from Appendix A)				
Flat	Test Member	SiO ₂	CaO	Total Cement Content
78	Ceiling 2	13.16	4.26	20.4
99	Ceiling 2	13.54	4.72	20.3

Table 3 - Sandberg cement composition test results

CEMENT COMPOSITION INTERPRETATION				
% by weight of cement (from Interpretation)				
Flat	Test Member	SiO ₂	CaO	Total (SiO ₂ + CaO)
78	Ceiling 2	64.5	20.9	85.4
99	Ceiling 2	66.7	23.3	90

Table 4 - Interpretation of Sandberg Cement Composition Testing

CEMENT COMPOSITION COMPLIANCE WITH BS EN 197-1:2011 (5.2.1)				
Flat	Test Member	Cement Consists of at least 2/3 (CaO + SiO ₂)	The Ratio of CaO / SiO ₂ > 2.0	
78	Ceiling 2	85.4 > 56.9 ∴ Pass	0.32 < 2.0 ∴ Fail	
99	Ceiling 2	90 > 60 ∴ Pass	0.35 < 2.0 ∴ Fail	

Table 5 - Comparison of interpreted results with BS EN 197-1:2011

By inspection of the interpreted results, the quantities of Silica (SiO₂) and Calcium Oxide (CaO) does not satisfy the expected proportions for today's standards.

The concrete mix therefore appears to have been inadequately mixed and is likely to cause negative impacts on the overall structural integrity of the structure and provide a less protective environment to the reinforcement.

Compressive Strength

In order to assess the robustness of the precast concrete elements forming St James's House, the characteristic compressive strength of the concrete was required. For the testing of hardened concrete, the method employed is to carry out core samples of representative areas of the block and subject the core samples to increasing compressive forces, within a laboratory, until failure.

The concrete cores taken from St James's House were from cross walls and flank walls [It was not possible to undertake core sampling of floor slabs as all flats within the block were single level (no duplex apartments) so coring through would impact another inhabited flat], each 100mm in diameter. These were sent to the Sandberg laboratory, and the compressive strength of each core determined. The results of the testing can be seen in Figure 13.

FLAT	SAMPLE	CORE LOCATION	COMPRESSIVE STRENGTH
Flat 16	1	Cross Wall 1	47.9 N/mm ²
Flat 16	2	Cross Wall 2	26 N/mm ²
Flat 16	3	Cross Wall 3	37.4 N/mm ²
Flat 78	4	Flank Wall 1	63.4 N/mm ²
Flat 78	5	Flank Wall 2	63.2 N/mm ²
Flat 78	6	Cross Wall 1	46.8 N/mm ²
Flat 99	7	Flank Wall 1	73.3 N/mm ²
Flat 99	8	Cross Wall 2	45.2 N/mm ²
Flat 99	9	Flank Wall 2	89.8 N/mm ²
Flat 99	10	Flank Wall 3	82.4 N/mm ²

Figure 13 – Compressive Strength results from the core samples taken in St James's House

Using the results obtained from the laboratory testing of each core, the characteristic compressive strength of the concrete could be determined. The calculation of the characteristic compressive strength was carried out in accordance with the method given in BS 6089:2010 – *Assessment of in-situ concrete strength in structures and precast concrete components*, and Concrete Advice No.47 – *Assessment of in-situ concrete strength using data obtained from core testing*.

However, it should be noted that the results have a large degree of variation between the cross walls and the flank walls (both flank wall cores tested to be significantly higher at averaging around 74 N/mm²). This may suggest that the flank walls were designed to have a higher concrete strength than the cross walls. Typically, in LPS blocks we have found that all wall panels (if not floor slabs as well) are all cast with similar concrete compressive strengths (although there usually is some discrepancy due to poor quality control during casting and compression hardening over the years). During the intrusive phase, based on the assumption all walls are typically cast with similar concrete grades, Ridge obtained only 2 flank wall samples (cross walls are more abundant within the block, and flat 16 contained no flank walls). It is therefore not possible to conduct a separate assessment on the flank wall results using the codes noted above due to the sample size. In the first instance, the assessment was therefore carried out ignoring the flank wall results and conservatively

adopting the calculated characteristic compressive strength of the cross walls for all loadbearing members, with the provision that if the latter disproportionate collapse assessment found only the flank walls to fail due to this reduced compressive strength, that further core testing could be carried out at a later date (ultimately found not to be required).

Based on the cross wall core results, show that the characteristic compressive strength of the concrete at St James's House is **21.3 N/mm²**. For flank wall characteristic compressive strength is **49.7 N/mm²**

5.5. Observations during the Intrusive Phase

During the intrusive investigations the following defects / observations were made on the construction of the block:

- There appeared to be poor quality control during the casting of the ceiling. The concrete cover to the embedded reinforcement appeared to vary considerably throughout the areas inspected. In some areas the cover to the reinforcement was noted as <10mm.

This will have consequences in terms of both risk of corrosion but may also present a risk to structural fire resistance in relation to the time for which the building structure can resist fire. We would advise Brighton and Hove city council review this with a suitably qualified Fire and Structural Engineer.

- Flat 78
 - Compaction in flank wall joint is poor, the bars have surface corrosion due to voids.
 - Remedial ties were observed – 25 \emptyset ribbed bars inserted diagonally from the floor into wall joint. However, this was only found on flank and spine walk. There were no remedial ties cross walls and ceiling.
- Flat 99
 - Hallway floor to floor joint transverse bars densely packed. There are voids in concrete resulting to corrosion.
 - Flank wall joint is poorly compacted.
 - Very low concrete covers. The minimum recorded, 4mm. Surface corrosion were noted.
- Flat 16
 - Generally adequate.

6. ST JAMES'S HOUSE STRUCTURAL ASSESSMENT

The findings of the on-site investigations were then used in the desktop study to justify the robustness of the block.

6.1. Assessment Criteria

The block has been assessed using the 2012 BRE Report 511 titled 'Handbook for the structural appraisal of Large Panel System (LPS) dwelling blocks for accidental loads'. The report identifies three criteria to assess LPS blocks against. The block needs only pass one of the following criteria:

- LPS Criterion 1. There is adequate provision of horizontal and vertical ties to comply with the current requirements for the relevant Consequence Class for each block as set down in the codes and standards quoted in Approved Document A – Structure as meeting the requirement set down in the Building Regulations.
- LPS Criterion 2. An adequate collapse resistance can be demonstrated for the foreseeable accidental loads and actions.
 - The blocks are currently fitted with a piped-gas supply, and as such the main structural members are to be assessed for an overpressure of 34kN/m².
 - Should the structure be shown to be inadequate for the above loading, the assessment shall be repeated for a reduced overpressure of 17kN/m² – this would be associated with a block without a piped-gas supply.
- LPS Criterion 3. Alternative paths of support can be mobilised to carry the load, assuming the removal of a critical section of the load bearing wall in the manner defined for Class 2b in Approved Document A – Structure or alternatively assuming the removal of adjacent floor slabs (taking the floor slabs bearing on one side of the wall at a time) providing lateral stability to the critical section of the load bearing wall being considered. (BRE, 2012)

The following sections document the main findings of the investigation and a summary of each LPS Criterion assessment.

6.2. LPS Criterion 1 – Adequation Provision of Ties

The first stage in the assessment to determine the adequacy of the joints is to define the 'Consequence Class' of the blocks. Based on the definitions provided Building Regulations Approved Document A the block falls into Consequence Classes 3. The block therefore requires effective horizontal and vertical ties. The details for the joints between floors and walls can be seen in Section 5.3.

The effectiveness of horizontal and vertical ties is assessed against the Eurocode document BS EN 1991-1-7:2006 Actions on Structures – General Actions – Accidental Actions.

Cross Wall / Floor Slab Joints

The assessment of the cross wall / floor slab joint has shown that:

Horizontal Ties: Ineffective

Vertical Ties: Ineffective

The cross-wall joint is therefore **insufficient** to pass the assessment for a Consequence Class 3 building.

Flank Wall / Floor Slab Joints

The assessment of the cross wall / floor slab joint has shown that:

Horizontal Ties: Ineffective

Vertical Ties: Ineffective

The flank wall joint is therefore **insufficient** to pass the assessment for Consequence Class 3.

Flank Wall (Corner Panel) / Floor Slab Joints

The assessment of the cross wall / floor slab joint has shown that:

Horizontal Ties: Ineffective

Vertical Ties: Ineffective

The corner flank wall joint is therefore **Insufficient** to pass the assessment for a Consequence Class 3 building.

Floor / Floor Joints (In Hallway with no Cross Wall)

The assessment of the cross wall / floor slab joint has shown that:

Horizontal Ties: Ineffective

Vertical Ties: Not required *

The floor joint within the hallway is therefore **Insufficient** to pass the assessment for Consequence Class 3.

* As there are no vertical members above this joint there is no need for a vertical tie

Cross Wall Vertical Joints

The longitudinal vertical joints between abutting wall panels is in the form of a 'shear key' connection. It would be expected that this type of joint would contain reinforcement, similar to that shown in Figure 14, to prevent the joint from uncontrolled separation.

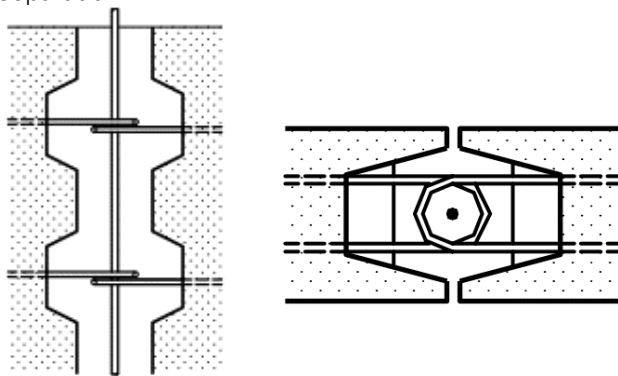


Figure 14 – Typical shear key connection at a vertical joint between wall panels (fib, 2008)

However, the U-bars protruding from each panel are in the other orientation and therefore do not connect 'loop around' the vertical lacer bar.

The joint therefore does not possess an adequate horizontal tie. (No vertical tie would be required in this type of connection for either Consequence Class)

Spine Wall / Spine Wall Joints

As per Cross Wall.

ST JAMES'S HOUSE (CONSEQUENCE CLASS 3)			
JOINT TYPE	ADEQUATE HORIZONTAL TIE	ADEQUATE VERTICAL TIE	NOTES
Flank Wall	x	x	Inadequate
Cross Wall	x	x	Inadequate
Wall to Wall Joints	x	x	Inadequate

ST JAMES'S HOUSE – LPS CRITERION 1 – ADEQUATE PROVISION OF TIES

FAIL

6.3. LPS Criterion 2 – Adequate Collapse Resistance

BRE Report 511 states that as the majority of elements in an LPS dwelling block are loadbearing they must be treated as 'key elements'. Collapse resistance calculations have been carried out for the block, based on the findings of the intrusive investigations carried out on each of the main loadbearing members.

The calculations have been carried out using British Standards which have been chosen as they are akin to the design codes that the structure would have been originally designed to, rather than the modern Eurocodes.

As there are no piped-gas supply within the flats calculations were carried out to assess the block of flats for a reduced over-pressure of 17kN/m². This over-pressure would relate to a block without a piped-gas supply. The calculations show that the structural elements that form St James's House are insufficient to resist a loading of this magnitude.

The following table summarises the findings:

STRUCTURAL ELEMENT	17KN/M ² OVERPRESSURE (WITHOUT PIPED-GAS SUPPLY)	NOTES
Floor Slab (Downward)	X	Inadequately Robust
Floor Slab (Uplift)	X	Inadequately Robust
Flank Wall	X	Inadequately Robust
Cross Wall	X	Inadequately Robust

– LPS CRITERION 2 – ADEQUATE COLLAPSE RESISTANCE

FAIL

6.4. LPS Criterion 3 – Alternative Load Paths

In the event of an explosion without a piped-gas supply, the bounding enclosure area would be a single room. The overpressure from such an event is considered to act on all elements within this bounding enclosure simultaneously. This could result in one floor slabs and two wall panels being subjected to this loading.

Due to the inadequacies in the joints and the robustness of the panels, it remains unlikely that the block would be able to mobilise alternative load paths in the event of an internal explosion. This may lead to the disproportionate collapse of the block. However, it should be noted that the risk of collapse will potentially be lessened by the removal of the gas.

Due to the inadequacies in the joints and the robustness of the panels, it is unlikely that the block would be able to mobilise alternative load paths in the event of an internal explosion of this magnitude. This may lead to the disproportionate collapse of the block.

ST JAMES'S HOUSE – LPS CRITERION 3 – ALTERNATIVE LOAD PATHS

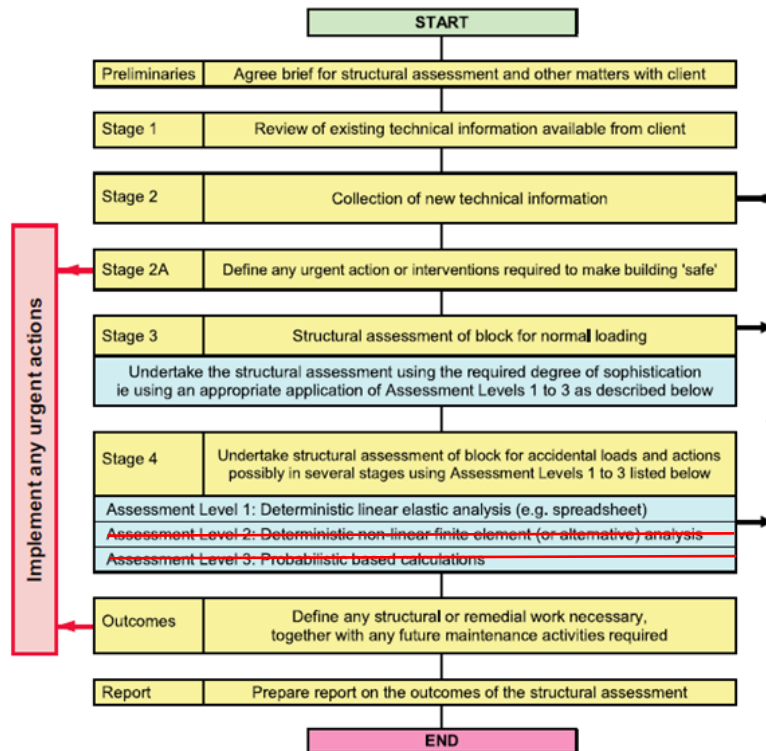
FAIL

6.5. Summary of assessment

There is not a piped-gas supply to the flats, reducing the risk. However, the block remains insufficiently robust to resist disproportionate collapse.

7. CONCLUSION

To carry out the assessment of the block the engineers at Ridge have carried out desktop studies, on-site investigation, and structural assessment calculations in the following procedure:



The outcome of the assessment is that the block in its current state is inadequate to resist disproportionate collapse.

The concrete testing has shown that:

- Within the precast concrete panels, the reinforcement is at varying levels of risk of corrosion. This is mainly due to high levels of chloride within the concrete. Minimal concrete cover within the floor slabs is also a contributing factor.
- Within some of the insitu joints, the reinforcement is actively corroding. In some areas the corrosion has been accelerated due to voids in the concrete from items such as rope and polystyrene.
- The precast elements have a characteristic compressive strength of 21N/mm². [Note this is based on core sampling of the cross walls only. The two flank walls tested were found to have a compressive strength significantly higher – although this will not alter the assessment for the flank wall. The floor slabs could not be cored due to occupied flats above/below.]
- The compressive testing appears to indicate an adequate degree of quality control in the concrete mix during construction, with compressive strengths reasonably consistent.

It has been shown that under 'normal loading' (i.e. load induced from day-to-day living in the flats) that the structural elements are acceptable (except for the deflection of the slabs – although there are no visible signs of distress). This allows the block to remain habitable, whilst works are carried out on the block to address the failings of the accidental assessment.

8. RECOMMENDATIONS

8.1. Piped-Gas Supply

Best practise for long term measures (i.e., longer than 6-12 months) would comprise:

- Removal of piped gas supply (lower ground level), to a point outside of the curtilage

To reduce the risk of disproportionate collapse, we recommend to adopting the following measures.

1. An immediate ban on the use of any gas cannister/bottles/cylinders being used or stored within the dwellings, along with a complete ban on any other potentially explosive substances.
2. Ban of portable gas cookers. They are views as high risk as they have the potential to be left on, causing a leak that might then be ignited, causing explosion and excessive pressures being applied on the structures.
3. Undergo a detailed assessment to understand the blocks resistance to fire.

8.2. Strengthening Works / Risk-Reduction Measures

For risk analysis refer to St James's Risk Management report and along with the strengthening drawings.

Risk reduction measures, other than the removal of the piped gas, may include 'administrative' measures such as the installation of CCTV cameras with the aim of preventing gas canisters or other highly flammable objects from being brought into the block, installation of a fire alarm systems and an updated fire strategy,

As part of the fire strategy undertake an analysis of the building in the fire condition with a suitably qualified Fire Engineer and Structural Engineer to assess its ability to resist fire given the low covers experienced during the robustness assessment.

Strengthening works will need to be carried out due to the failure of LPS assessment. For further information on the elements that will require strengthening, refer to the strengthening drawings.

Action will also be needed to remediate the risk of corrosion to the block, as it was observed some of the reinforcement is now actively corroding for example. Flat 99. This may include anti-carbonation paint, or cathodic protection.

Finally, if the risk reduction measures cannot control the risk to acceptable levels, and the investment into strengthening works proves uneconomically viable, demolition may be a third option for the block.

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