RIDGE

ST JUDES – JOHN COZENS
STRUCTURAL ROBUSTNESS ASSESSMENT

BRISTOL CITY COUNCIL
November 2024

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STRUCTURAL ROBUSTNESS ASSESSMENT

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1. EXECUTIVE SUMMARY

Intrusive investigations were conducted on the dwelling blocks at St Johns, Bristol to verify their condition and construction. An assessment of their robustness against accidental loading and susceptibility to progressive collapse has also been carried out. The investigations showed that the building is a cross-wall system consisting of precast internal concrete walls with cast in-situ flank walls for stability. The floor slab consists of precast concrete gothic beams. The findings from intrusive investigations suggested that the building may be susceptible to disproportionate, progressive collapse and does not currently meet the disproportionate collapse requirements set out in Approved Document A. The building has been assessed as a consequence class 2b (upper risk group) because the building exceeds 4 storeys.

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

The building was assessed against BRE Report 511 which states that LPS blocks can be assessed under three criteria, of which a block needs only pass one. The criteria and results relating to John Cozens are as follows:

Table 1 - Summary of LPS Criteria for John Cozens

LPS CRITERION	ASSESSMENT	COMMENTS
Criterion 1 Adequate ties within joints	**Insufficient**	John Cozens is a class 2B building that requires both vertical and horizontal ties. The horizontal ties were found to be insufficient to withstand the required imposed forces due to inconsistencies in their installation. Investigations also found no adequate vertical ties between wall panels.
Criterion 2 Adequate strength to resist Accidental Loads without the steel frame	**Insufficient**	The floors cannot resist the overpressure requirement for non-piped gas supply of 17kPa. The wall panels appear to be satisfactory up to level 2.
Criterion 2 Adequate strength to resist Accidental Loads with the steel frame	**Partially Insufficient**	The floors can resist the overpressure requirement for non-piped gas supply of 17kPa, due to the steel frame strengthening being utilised. The wall panels pass up to level 2 given the additional weight of vertical loads above them; above this level the vertical load is insufficient to assist in resisting the flexure they experience.
Criterion 3 Ability to mobilise alternative load paths	**Insufficient**	The use of alternative load paths is not considered to be feasible, as each element is deemed critical to the system's integrity. The connections between elements are best described as flexible, with joint stiffness playing a role rather than functioning as true pin connections. Consequently, any failure within the system is likely to trigger a mechanism, leading to disproportionate collapse.

Table 2 - Assessment criteria summary for John Cozens

ASSESSMENT CRITERIA	ASSESSMENT	COMMENT
Fire Resistance Without the steel frame	**Insufficient**	A load bearing capacity of 60 minutes is calculated for the structure; the critical element considered is the floor which has a low reinforcement cover. A 90-minute requirement is needed as set out in current guidance.
Fire Resistance With the steel frame assuming fire protection	**Sufficient**	An estimated fire resistance of 90 mins can be achieved provided the steel frame is fireproofed to a similar effect.
Carbonation Depth of carbonation into concrete	**Insufficient**	Carbonation testing indicates that, in some areas, the passivity front has surpassed the reinforcement, and the concrete is at risk of spalling due to the corrosion and expansion of the steel reinforcement.
External Walls External masonry wall support and tie details	**Insufficient**	The external masonry walls on the building, consisting of two layers of blockwork, were found to be inadequately tied to the primary concrete walls and floors. With improper ties the masonry panels pose a risk of collapse in high wind loads.
Balustrades Condition of metal balconies	**Insufficient**	Balustrades around the building, particularly along the shared access walkways, were noted to be severely corroded with several instances of temporary propping being used to support the balustrades. The condition of the balustrades requires replacement of the full system.
Balconies	**Insufficient**	The visual condition of the concrete balconies, combined with carbonation results indicate that carbonation level is high increasing the risk of spalling. The increased risk category of the nursery below should be considered as part of any following risk workshops.

In addition to the inspection and assessment of the concrete frame, visual surveys of the overall building condition was carried out. Areas reviewed include the external wall cladding, handrails and balconies. It was found through the intrusive investigations that the masonry infill panels that span between the structural concrete frame have very few walls ties both between the cavities and back to the structural frame. The balconies to the south side of John Cozens were noted to have spalling concrete with incidences noted where sections of the balconies had collapsed.

The foundations have not been specifically reviewed but no adverse movement has been noted during the investigation and therefore this suggest the foundations are performing adequately at this point in time. To mitigate any long-term risks of the foundations degrading primarily against chemical attack, further investigations of the footings could be completed.

Recommendations

Considering the above results of the assessment & the general condition of the block, our recommendations for risk reduction measures are as follows:

Immediate Term (0-6 Months)

1. Continuation of the updated building evacuation strategy to a simultaneous evacuation, with the continued waking watch across St Jude's. This is a short-term measure in line with Government guidance (Evacuation guidelines for fire and rescue services (accessible))
2. Installation of fire detection and alarm system (BS5839 - 1 Cat L5) to replace waking watch in accordance with NFCC guidance
3. Regular inspections for and immediate ban on:
    a. any gas cannister/bottles/cylinders being used or stored within the dwellings, along with a complete ban on any other potentially explosive substances (including high-capacity batteries which may be found in items including e-scooters/e-bikes and some newer models of mobility scooters).
    b. Portable gas cookers – viewed as high risk as they have the potential to be left on whilst unignited, causing a leak that may then be unintentionally ignited, causing an explosion and excessive pressures being applied on the structures.
    c. To limit hoarding to minimise fire loads in flats
4. Removal of gas supply to laundry rooms and presence of diesel generators near the building that could increase the risk of an accidental loading scenario.
5. Full condition survey of the balustrades around John Cozens, temporary support provided to those in a critical condition with a design and programme developed to replace all the balustrades.
6. Detailed condition surveys of the balconies and walkways due to carbonation of the concrete to identify deteriorated and degraded areas or the structure to enable repairs as necessary.
7. Erection of the non-combustible scaffold fan to the base of the block to prevent falling concrete.
8. Detailed wind analysis of the block to be undertaken to assess peak forces on the external masonry wall with remedial design / strengthening options.

Medium Term (6 months -2 Years)

1. Installation of sprinkler protection to BS 9251 Category 4 and conversion of existing detection system, or enhancement of the fire protection of the structure to increase the fire resistance.
2. Repairs to concrete on residential balconies and communal walkways and Removal of residential balconies.
3. Carry out an options appraisal to understand the cost benefit of upgrading the structure to resist disproportionate collapse then:
    a. Upgrade the structure through ties or strengthening to resist disproportionate collapse forces and provide a robust structure.
    b. If strengthening works are unviable re-assess the risk measures in place and determine any further measures that will enable the block to remain in service over a short term until decant can be undertaken for demolition.
4. Repairs and or replacement of the residential balconies due to deterioration from carbonation.
5. Remedial repairs to the escape walkways following detailed surveys.
6. Remedial repair works to the external masonry wall, or overclad the existing envelope.
7. If the block is to be retained investigate and assess the foundations for deterioration and chemical attack.

Long Term (3-5 years+) Continued Inspections

Considering the buildings type and height the following recommendations are made, which align with BRE recommendations:

a) A programme of visual inspections at intervals of 1 year, 2 years and 5 years following this initial appraisal, and then every 5 years subsequently to the external envelope (including parapets and balconies) to identify potential hazards from falling debris.

b) Visual inspections at 10-year intervals to structural joints which are vulnerable to water penetration; locations such as flank walls and roofs.

c) Full appraisal of the whole building at 20-year intervals

Should the risk reduction measures fail to effectively control the risk of disproportionate collapse to acceptable levels, and investment into strengthening works prove uneconomically viable, demolition of the block might be considered as a final long-term approach for the block. However, we would recommend that this decision should only be taken following the completion of a remedial strengthening design review, supported by the risk and cost benefit analyses recommended above to ensure that demolition is the best approach.

2. INTRODUCTION

2.1. Site Address

St Jude's
Great Ann St,
Bristol
BS2 0DX

Figure 1 – St Jude's Location (Google Maps, 2024)

2.2. Structural Engineering Brief

Ridge and Partners LLP (Ridge) were appointed by Bristol City Council to undertake a combination of visual and intrusive surveys to assist with provision of information for the Building Safety Case and Risk Assessment of multiple dwelling blocks at St Jude's, Bristol. These include John Cozens, Haviland House, Langton House and Charlton House.

The brief was therefore to carry out an audit on the construction of each block, based on available historic information, followed by detailed intrusive investigations into selected areas of the block. The construction details of the block, gathered from this audit, will serve as the foundation for a structural assessment. This assessment will evaluate whether the block has sufficient capacity to resist progressive collapse in the event of an accidental incident.

2.3. Report Contents

The contents of this report relate exclusively to the construction of John Cozens 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. Refer to Appendix A for the detailed testing results of John Cozens.

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

2.4. Limitations

Throughout the duration of the intrusive investigations the blocks remained inhabited by residents, with health and safety measure put in place including temporary relocation of residence, monitoring of disruption and provision of personal protective equipment (PPE). This presented challenges to the investigation team in terms of availability of vacant flats within which intrusive investigations could be undertaken. Four suitable flats were identified, namely flats 17, 18, 33 & 43.

Whilst the investigative works were detailed, with multiple tests carried out in each of the four flats, it should be noted that many areas of the block were not tested and thus the assessment of the blocks can only be based on what was uncovered in the sample investigation. The investigations were also only carried out from within the flats, with the exception of localised core samples which used the shared access walkways to gain a wide range of sample locations.

All flats within the John Cozens block are 2-bedroom maisonettes, with the floor constructed from a series of precast concrete gothic beams. It was therefore not possible to obtain core samples from floor slabs for compressive testing due to the nature of the construction of the floor slab.

2.5. Purpose of report

The purpose of the report is to advise on the cross-wall construction of John Cozens and its susceptibility to disproportionate collapse, together with an assessment on the condition of the building, and it is not intended to be used for any other purposes. This report is for the sole benefit of the client and may only be used 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 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.

3. BACKGROUND INFORMATION

3.1. General Building Information

John Cozens is one of 4 inter-connected residential blocks within the St Jude's estate. This report only considers the assessment of John Cozens.

The dwelling block, John Cozens located in St Jude's, Bristol was assessed for its robustness to resist accidental loading from over-pressure, such as an internal gas explosion, and its susceptibility to progressive collapse. The block is believed to have been constructed by Stone, with J. Nelson Meredith Architects from a form of precast concrete construction for Bristol City Council, with construction commencing in 1957.

Figure 2 - St Jude's Layout (Goole Maps, 2022)

Ridge & Partners LLP were able to access the record files held by Bristol City Council which provided some basic details of the construction sequencing of the blocks and give some indication of construction details. It appears the blocks were constructed in two stages with Charleton and Langton Houses (Blocks A & B) being built as part of Stage 1 and Haviland House & John Cuzon's being built as part of Stage 2.

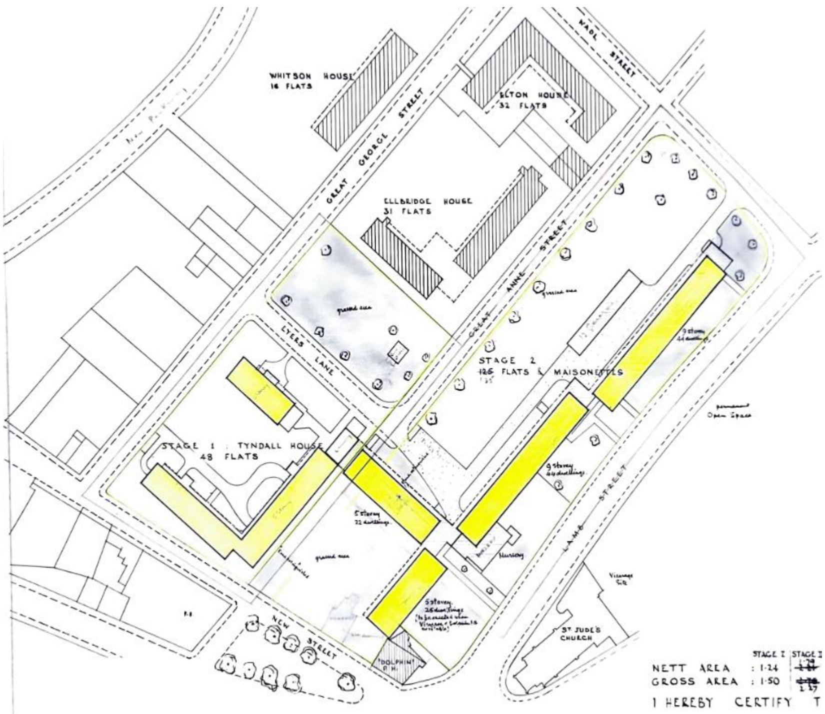


Figure 3 - As built site plan of Blocks A, B & C

The flats in John Cozens are two storey maisonettes with a lower floor containing the kitchen & living room and two bedrooms and a bathroom on the upper floor.

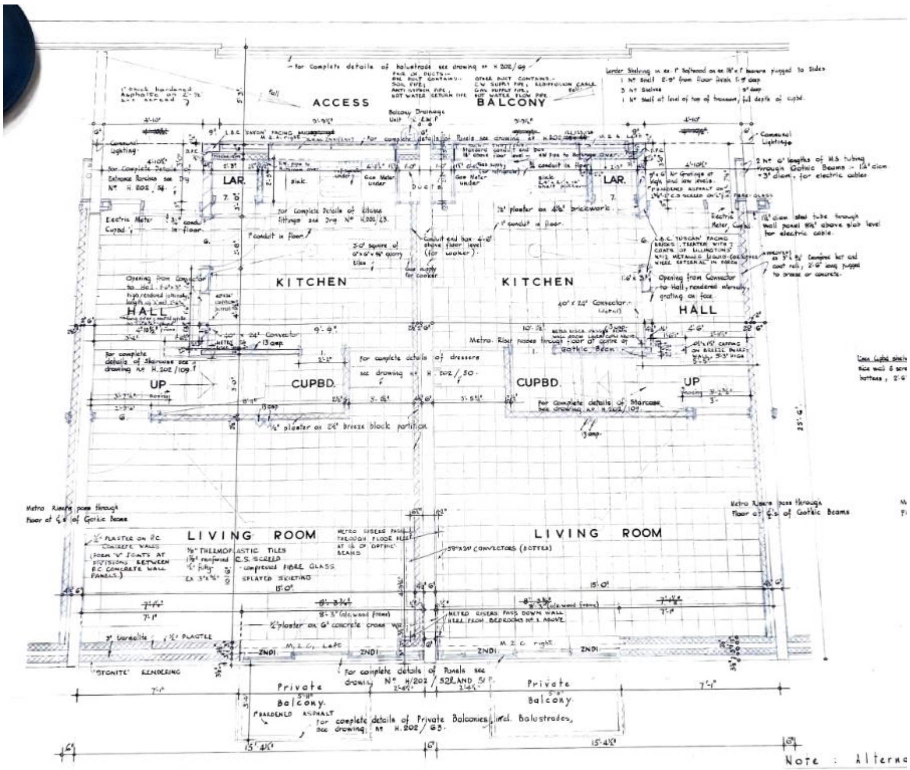


Figure 4 - Haviland House & John Cozens Lower floor record plans

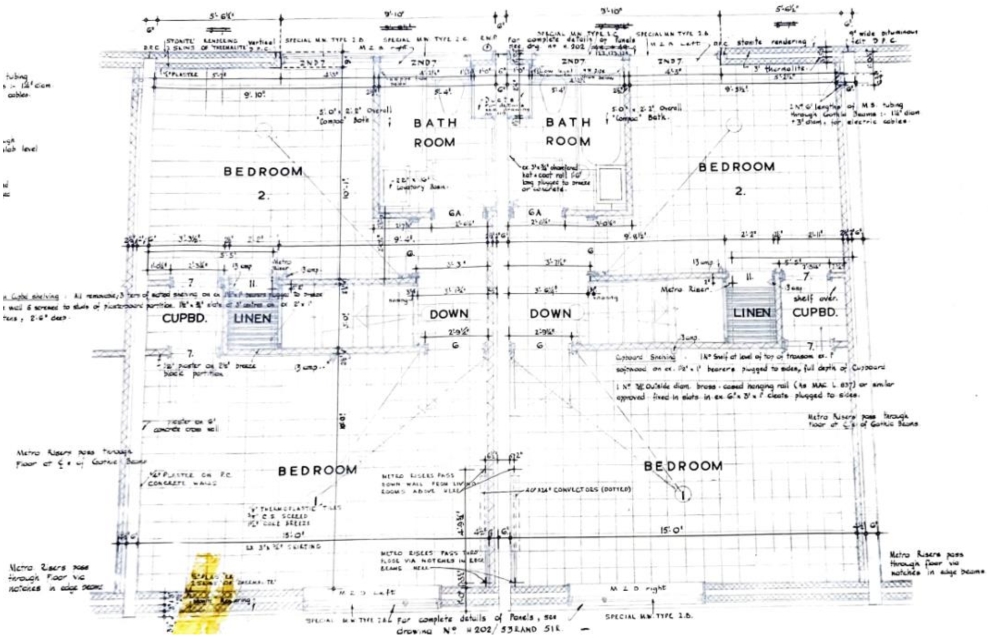


Figure 5 - Haviland House & John Cozens upper floor record drawings

Each flat in John Cozens has an internal staircase that that runs up through the centre of the flat. The opening forming the stairs is trimmed by a pair of precast concrete beams as indicated on the record drawings. The trimming beams were exposed and confirmed as part of the site investigations. The beams appear to share a bearing on to the precast cross walls. Access to the bearing of the beams was not possible during the investigations carried out and as such the condition and connection detail was not confirmed on site.

Access to each of the flats is via a shared walkway along the front of the building formed by a cantilever section of the precast cross walls, which supports a continuation of the gothic beam floors. The access walkways are accessed via a lift & stair core at either end of John Cozens where it connects with the adjacent blocks.

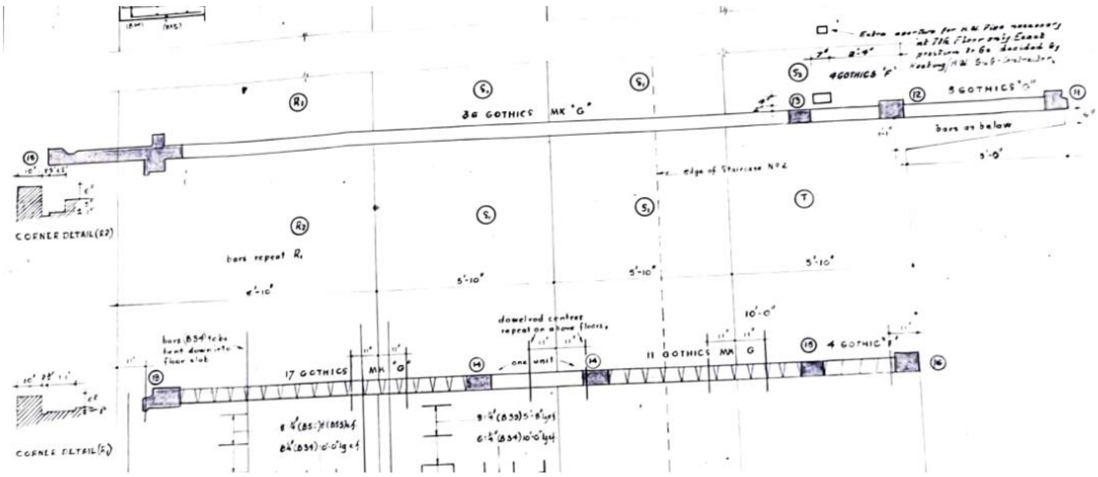


Figure 6 – John Cozens typical record drawings section.

Each flat has a balcony accessed from the lower-level living area extending approximately 1000mm off the rear elevation of the building. The balconies were noted as being in a poor condition with concrete spalling noted throughout and sections of concrete have anecdotally been reported to have been detached from the frame in some isolated areas. The balconies connect onto the precast concrete edge beam that spans between the cross walls. Due to the torsional forces induced by the balcony the investigation of the bearing of this edge beam was not possible as we did not want to adversely affect the stability of the beam through removal of a section of the cross wall at the bearing. However, GPR scans were completed on the area appear to indicate an additional bar below the beam. Therefore, it is assumed that the precast edge beams and balconies are held in position with a central locating dowel and through the friction from the self-weight of the precast panels. A review of these edge beams appears to have been completed previously with the introduction of some retrofitted strengthen in the form of steel angles fixing between the edge beams and precast panels.

3.2. History of LPS Blocks and Disproportionate Collapse

On 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.

Investigations and testing were undertaken on the remaining structure, focusing on the key structural elements and their associated joints to determine their strength. Following the investigations, the Tribunal made several recommendations. These included strengthening works required specifically on Ronan Point, but also recommended actions to be taken on other LPS structures. Existing LPS structures were required to be appraised and strengthened as required, and proposed LPS blocks were to be designed to resist disproportionate collapse.

Following the investigations, the Ministry of Housing and Local Government (MHLG) issued MHLG Circulars 62/68 and 71/68 titled '*Flats constructed with precast concrete panels. Appraisal and strengthening of existing blocks: Design of new blocks*'. The circulars outlined the recommendation that all blocks over six storeys in height were to be appraised by a structural engineer to determine whether the blocks were susceptible to progressive collapse. Two methods were outlined in MHLG Circular 62/68 to prevent progressive collapse in LPS blocks. Method A was to provide alternative load paths should a critical section of a loadbearing wall be removed. Method B was to ensure the structure had sufficient stiffness and continuity to resist the over-pressure loads. For Method B the circular stated that an over-pressure of 5 lb/in^2 (34kN/m^2) should be taken unless actions were taken to control the risk of explosion where a reduction could be made. MHLG Circular 62/68 also stated that tensile resistance could be achieved between panels by either welding together the projecting reinforcement or by loop bars projecting from each panel which were tied together using longitudinal dowel bars.

Following the publication of the circulars the Institution of Structural Engineers published Report RP68/02 titled '*Notes for guidance which may assist in the interpretation of Appendix 1 to MHLG Circular 62/68*'. The report included a recommendation that if the dwelling blocks did not have a piped gas supply, the over-pressure used in Method B of MHLG Circular 62/68 could be reduced to 2.5 lb/in^2 (17kN/m^2).

In 1970 the Building Regulations were updated to include Section D17 regarding provisions to resist progressive collapse. The new section reduced the number of storeys required for an assessment to be carried out on a block to five storeys or more (a more normal Government definition of 'high-rise'), representing a reduction of two storeys from that stated in MHLG 62/68. However, the MHLG Circulars, specifically addressing LPS blocks, were not superseded by the new Building Regulations, nor changed/updated to reflect the reduced number of storeys. It is therefore believed that there was confusion over which code governed for LPS blocks. As a result, it is possible that many blocks between five and six storeys were not assessed for disproportionate collapse.

BRE Report 107: Part 2 produced in 1987 provided non-mandatory guidance on the assessment of LPS blocks. This included methodology for inspection of the joints between elements and procedures to evaluate the findings. This report also confirmed the requirement to assess all LPS blocks over four storeys, bringing this in line with Section D17 of the Building Regulations. The latest requirements for disproportionate collapse are defined in Building Regulations Approved Document A – Structure. This document divides building usage types into consequence classes, with differing levels of assessment required for disproportionate collapse. The consequence class table can be seen in Section 6.1.

BRE have also published an additional guidance document, Report 511 titled '*Handbook for the structural appraisal of Large Panel System (LPS) dwelling blocks for accidental loads*'. This report provides structural engineers with the methodology required to assess LPS blocks and summarises and documents the research the BRE have undertaken since the collapse of Ronan Point. This report has been used as the basis for our assessment of the blocks of flats at St Jude's.

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.

4. INSPECTION & SURVEYS

4.1. Methodology of assessment

The method used to carry out the assessment on John Cozens follows the hierarchical approach adopted by BRE 511 as shown in Figure 7 below.

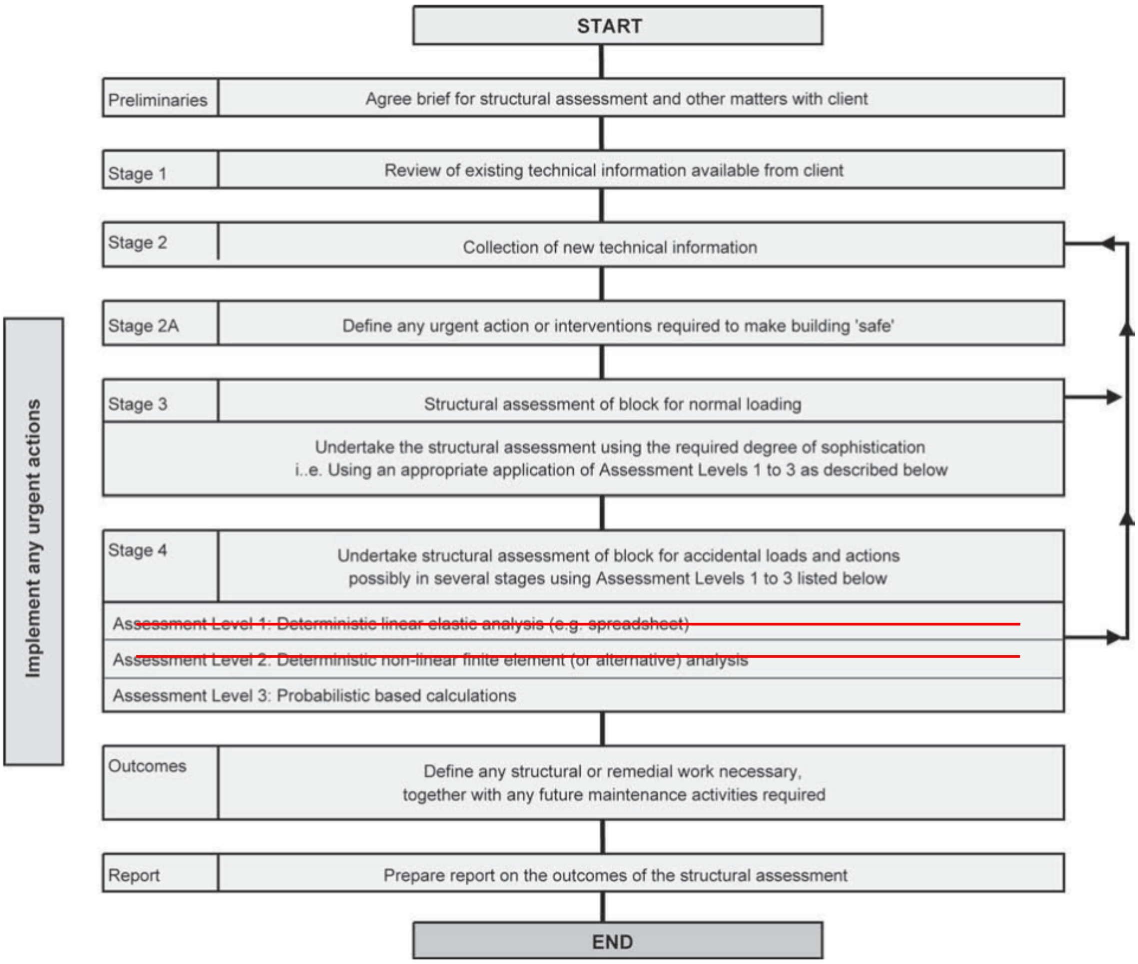


Figure 7 - Extract of BRE 511 figure 34 'Main steps in the structural assessment process'.

Four flats were identified in order to carry out the investigations with the aim to provide a suitable sample to cover most of the critical elements within the structure. Flats 17, 18, 33 and 43, were subjected intrusive breaking out to confirm structural details, this gave a sample of 9% of the total number of flat in John Cozens.

Using the limited As-Built information obtained from the construction details of the blocks Ridge subjected the four selected flats for both intrusive and non-intrusive investigation works to confirm the building's construction, including:

- Visual Inspection
- Concrete Reinforcement Scanning (Ferro & GPR)
- Concrete Testing (Insitu & Laboratory)
- Intrusive Opening Up Works

4.2. Observations during Intrusive Investigation Phase

The floor plans shown in Figures 8 & 9 highlight the locations of the investigations undertaken within the block. Flats 17, 18, 33 and 43 were subjected to the intrusive opening up works.

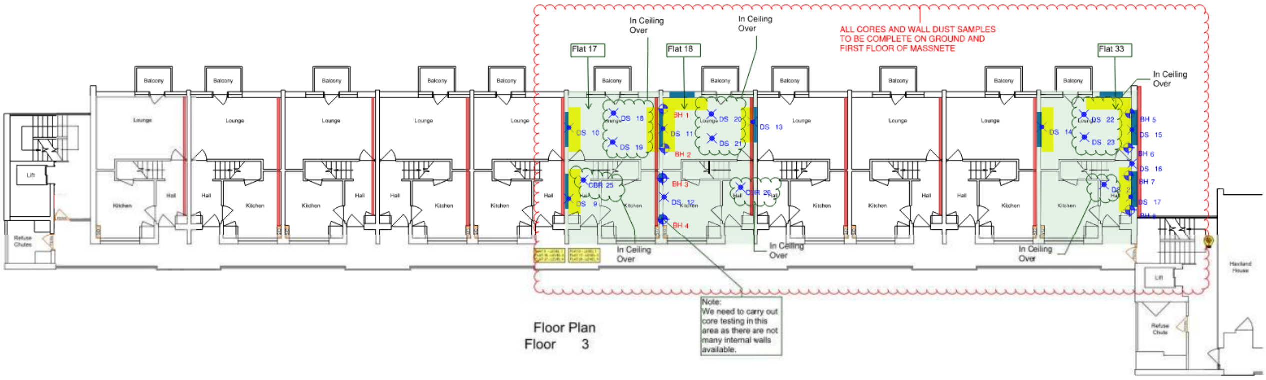


Figure 8 – Third floor plan for John Cozens showing the flats inspected.

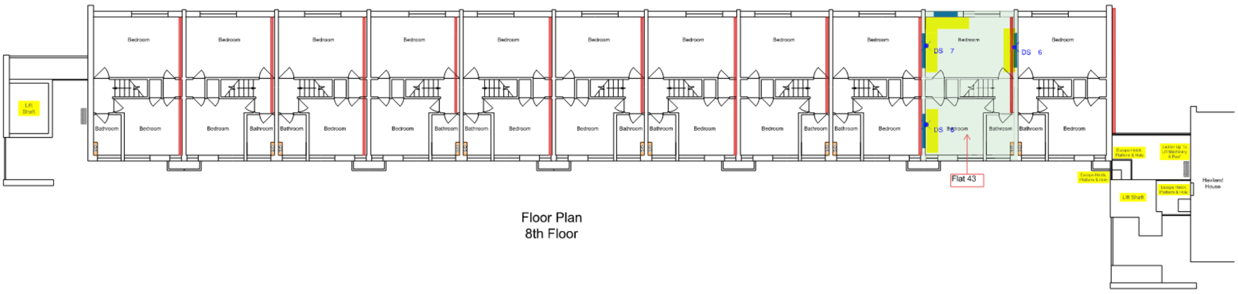


Figure 9 – Fifth Floor Plan of John Cozens showing the flats inspected.

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

- Poorly quality concrete to gothic beams in the sofit of multiple flats but most notably flat 18. Poor compaction can be seen by the void space in the beams.

Figure 10 – Individual gothic beam floor found to be of poor quality with poor compaction widely noted.

- Missing or unused masonry ties joining the masonry infill panels to the structural precast & insitu walls.

Figure 11 - No wall ties installed between blockwork and cross walls – Flat 18.

- Heavily deteriorated handrails with extensive oxidisation and corrosion at the connection to the shared balconies and walkways in many areas. It is evident temporary support has been installed prior to our commission to reduce the risk of failure of the balustrade.

Figure 12 - Corroded balustrade connection with temporary support installed.

- Spalling reinforcement on the individual flat balconies was noted in several locations.

Figure 13 – Poor condition to underside of balcony

4.3. Non-intrusive investigation findings

In addition to the intrusive investigation works a range of methods were used to identify the reinforcement in various structural elements. A mixture of Ferro scanning and Ground Penetrating Radar (GPR) was used to provide detailed scans of key elements.

- Precast concrete cross walls:
  Cross walls were shown through GPR scans to have 2 layers of reinforcement at 300mm c/c. The GPR scans also indicated that reinforcement was not continuous or linked across panel joints.

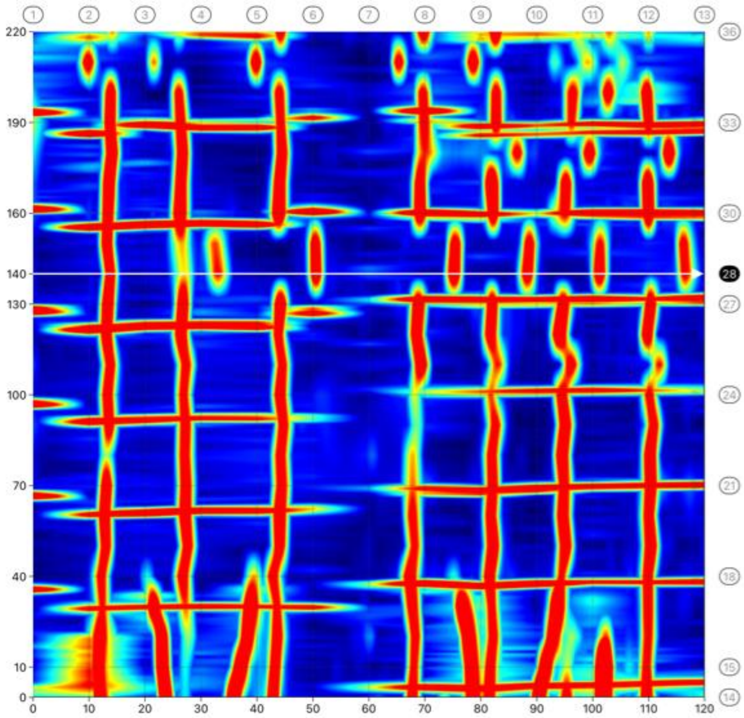


Figure 14 – Sample of GPR Scan results for a precast cross wall in John Cozens

- Through Ferro scanning the diameter of the reinforcement was indicated to be 6mm with the spacing confirmed as 300mm c/c.

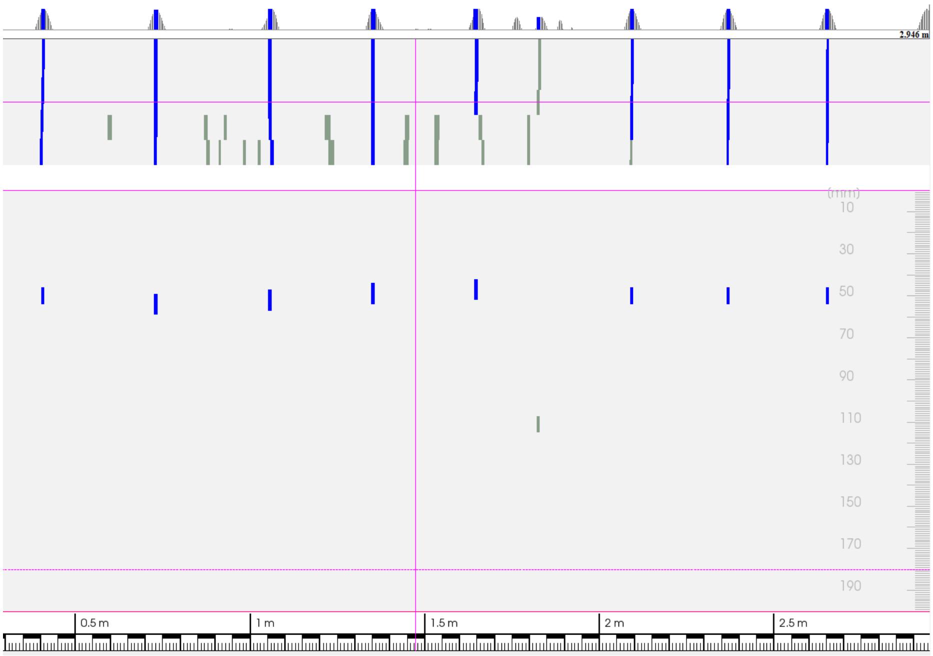


Figure 15 – Sample Ferro scan results of precast cross wall

- Intrusive investigations proved the diameter and spacing indicated on the two scans completed prior. Investigation at the panel joints also shows no reinforcement between panels highlighted by the GPR scan.

Figure 16 – John Cozens precast panel joint and exposed reinforcement with blockwork ties left in position from casting and not pulled out for use.

- Scans of the gothic beam floors with the ferro scanner identified the bottom reinforcement size, spacing and diameter.

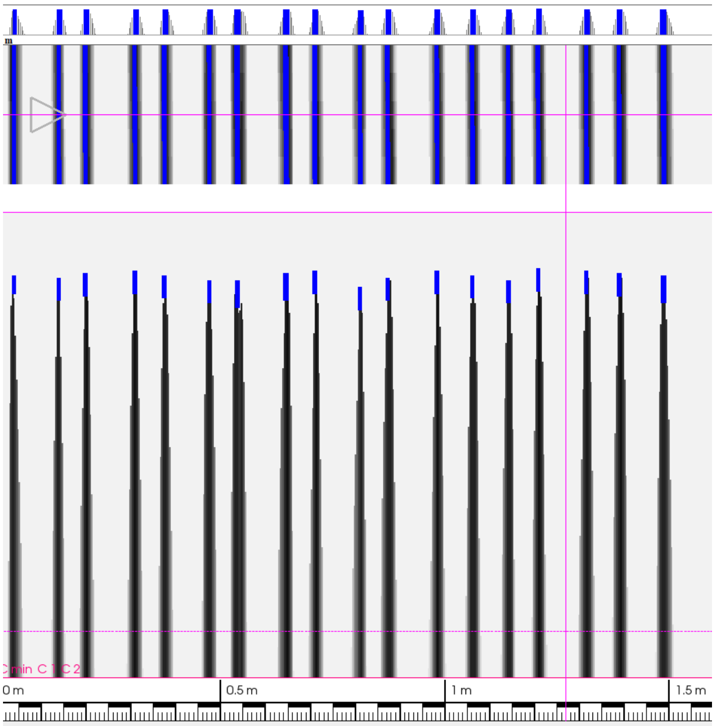


Figure 17 - Ferro scan of John Cozens floor soffit

4.4. Key Element Construction

The following section outlines the construction of key load-bearing elements within the structure. Almost all elements within a cross wall construction dwelling block can be considered as load bearing or contribute to the stability of the block. Below are the elements considered to be 'key elements'.

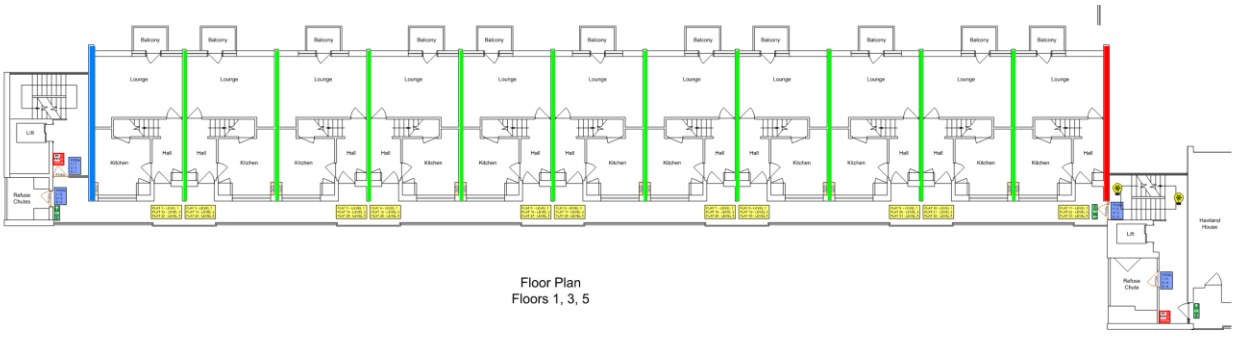


Figure 18 – John Cozens typical floor layout showing "key element" locations

The key elements, herein referred to as In-situ Flank Walls (shown in Blue), Precast Cross Walls (Green), Precast Flank Walls (Red) and Floor Slabs, were investigated to understand the construction. The construction of each is outlined below, with details of the embedded reinforcement and any notes against the element for variations in construction observed during the scanning & intrusive investigations.

Insitu Flank Wall Construction (Shown in Blue)

Height: 2.465m
Construction: 178mm thick concrete loadbearing wall panel
Reinforcement: Two layers 6mm square twist bars (300mm mesh)
Vertical Tie Reinforcement: 6mm bars @ 300mm c/c

Precast Flank Wall Construction (Shown in Red)

Height: 2.465m
Construction: 152mm thick concrete loadbearing wall panel
Reinforcement: Two layers 6mm square twist bars (300mm mesh)
Vertical Tie Reinforcement: None Present.

Precast Cross Wall Construction (Shown in Green)

Height: 2.465m
Construction: 152mm thick concrete loadbearing wall panel
Reinforcement: Two layers 6mm square twist bars (300mm mesh)
Vertical Tie Reinforcement: None Present.

Floor Slab Construction

Span: 4.686m (15′) max.
Construction: 171mm thick, Gothic beams at 165mm
Bottom Reinforcement: 10mm ($^3/_8$″) ribbed bars @ 110mm c/c
Tie Reinforcement: 10mm ($^3/_8$″) square twisted bars @ 330mm c/c (900mm embedment into floor either side of wall. Ties are missing in many locations)

Flank Wall and Cross Wall Joint Detail

The following annotated details illustrate the findings of the intrusive investigations for the various joint details between the load bearing members:

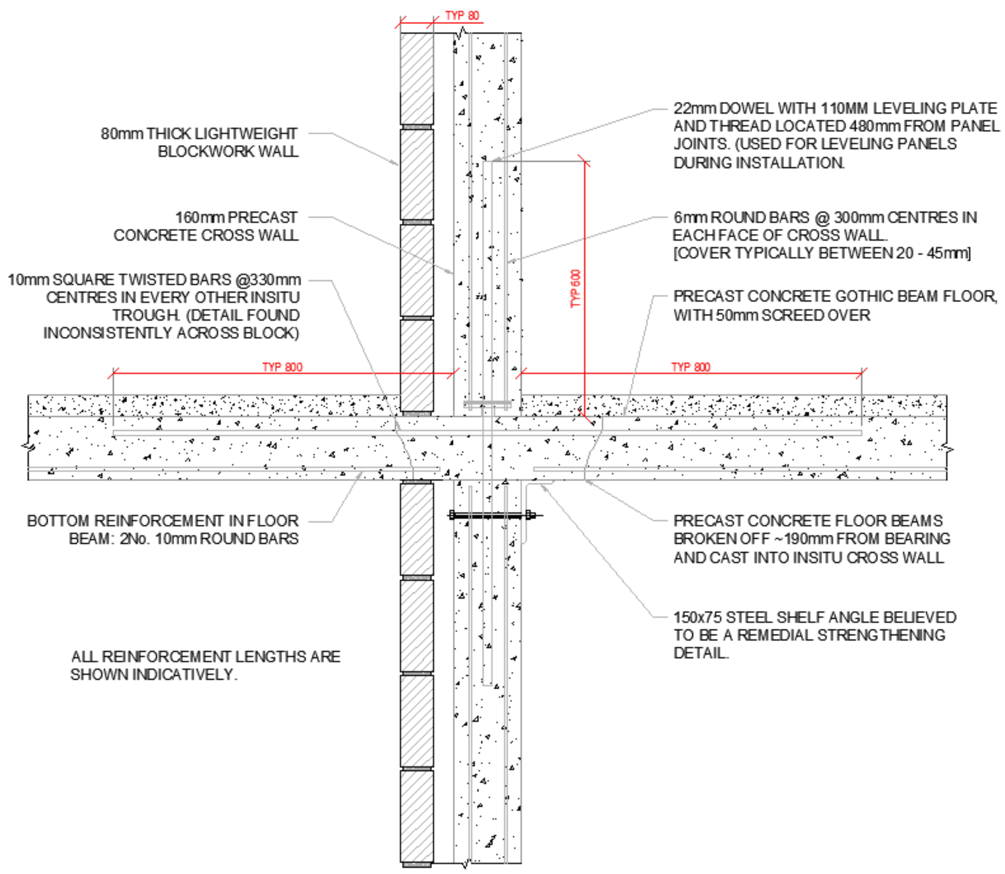


Figure 19 – Cross wall / floor slab joint

The tops of the floor beams were noted to have been broken off at their ends and had been incorporated into casting of the walls, likely to ensure a degree of homogeneity and to allow ties to be created between the elements.

The reinforcement within the wall panels does not appear to be continuous from panel to the next, being a precast concrete panel system, vertical ties should have been provided between the two precast panels in the form of cast in situ dowel bars. Vertical ties were not observed during the investigations suggesting no vertical tie is provided. A 22mm diameter dowel extending 600mm, is located in the centre of each panel 19" from the panel joints with at 110mm diameter levelling plate to help locate the panels during installation. The levelling plate has a thread on the bottom allowing for vertical adjustment of the panels. The levelling plate and dowel appear to have been left un-grouted and as such does not provide any vertical continuity between the two panels.

Record drawings suggest that horizontal ties should have been provided in every trough between the gothic beams, the horizontal ties found were found to be inconsistently installed across the block. In the locations where the spacing exceeds 600mm c/c the assumption for this assessment is that the tie is insufficient to resist disproportionate collapse.

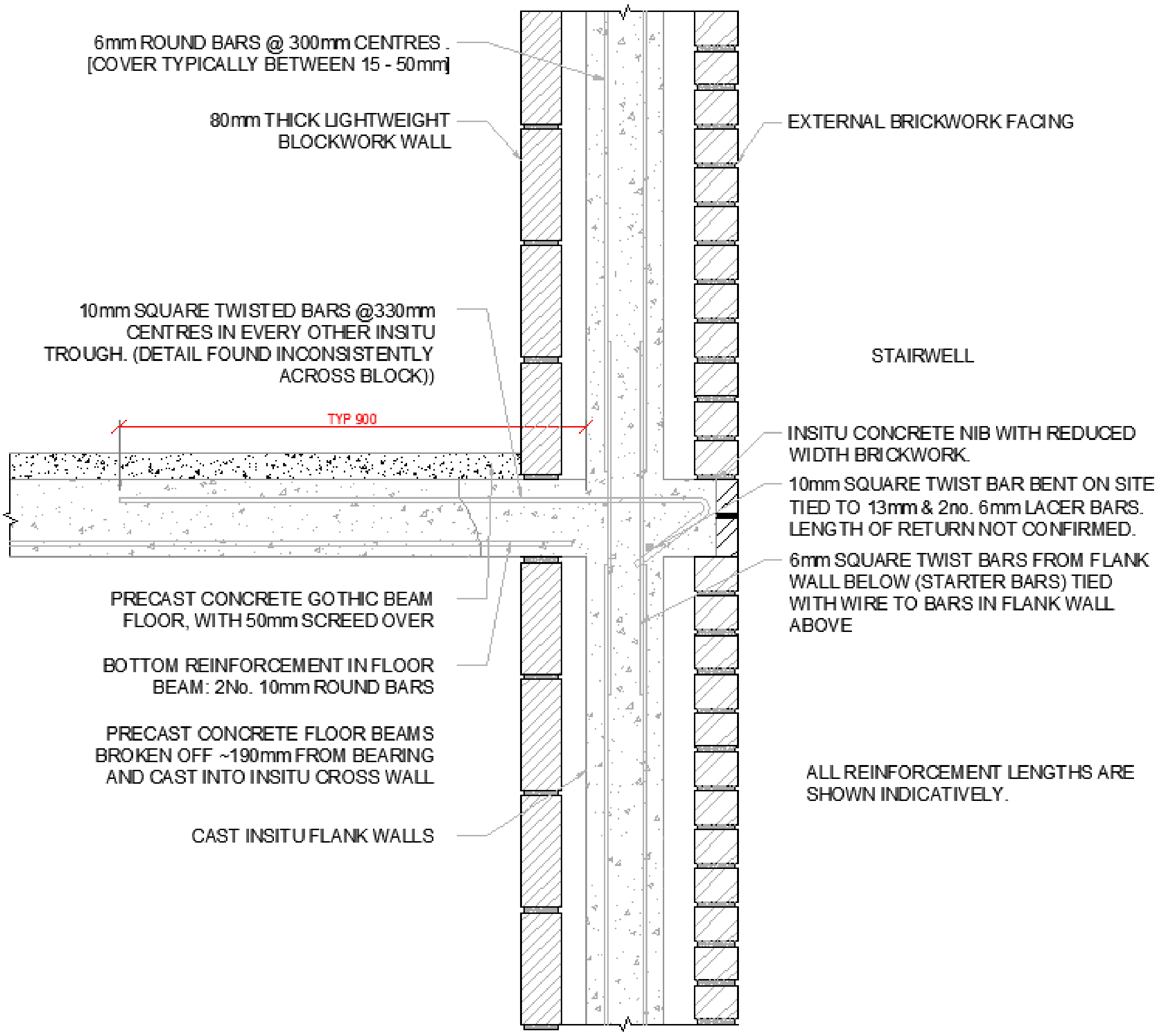


Figure 20 – Typical In-situ flank wall to floor slab joint

The investigations into the flank walls show that one flank wall is in-situ reinforced concrete and the other is precast as highlighted in Figures 18. The flank walls were found to blockwork inner leaf to the inside face of the flank wall, likely as a previous remedial action.

The investigation into the in-situ flank wall / floor slab joint suggested the existence of horizontal tie provision with 10mm square twisted bars provided in every other insitu trough being bent down into the wall panel. The bars were found on the back side of the wall and shown to tie to 2no. 6mm and 1no. 13mm longitudinal bar.

The investigation into the precast flank wall / floor slab joint suggested the existence of horizontal tie provision with 10mm square twisted bars are provided at varying centres and it was found that these bars bend down and return into the insitu stich between precast panels, linking around wall reinforcement and an additional 13mm lacer bar to provide an effective tie.

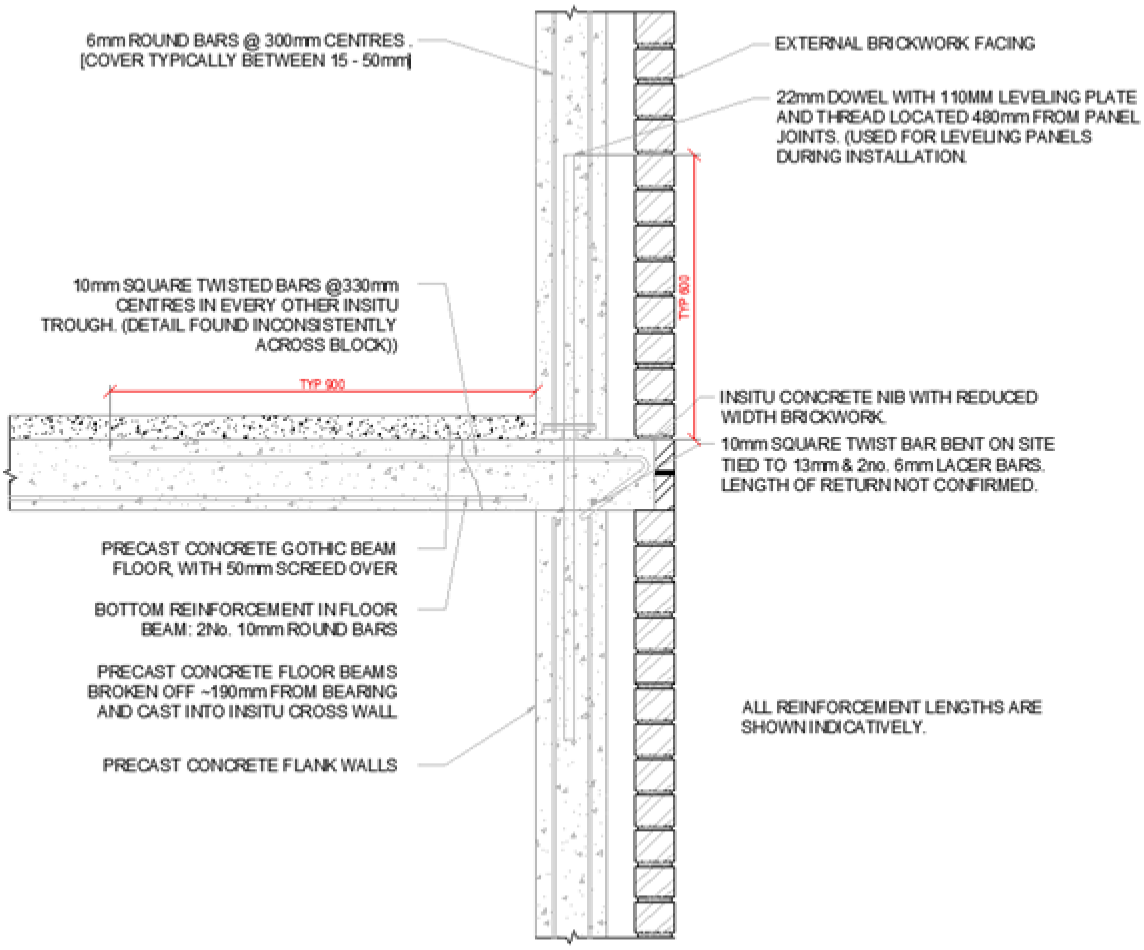


Figure 21 - Precast concrete flank wall/ floor slab joint

Wall Panel Vertical Joint

The following detail shown in figure 21 is a typical in-situ vertical joint between wall panels. Where a lacer bar(s) is located within overlapping u-bar tie reinforcement bars extending from each wall panel to provide a tie between the two panels.

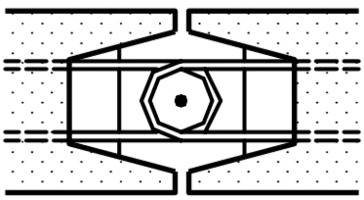


Figure 22 - Typical shear key connection at a vertical joint between wall panels

The existing John Cozens drawings indicate that no reinforcement was intended for the panel joints with only an insitu stich between panels. This detail was identified on site, with a grouted pocket between panels and no overlapping reinforcement – as shown in Figure 22 below.

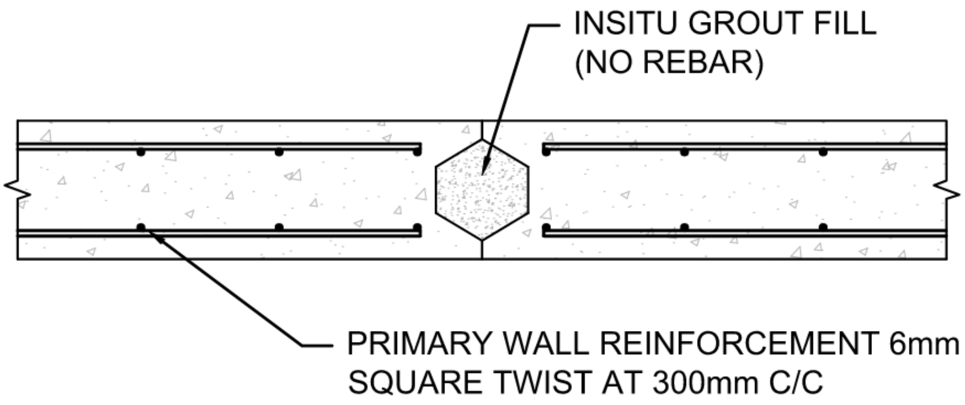


Figure 23 - Joint detail between cross wall panels as identified on site and in record drawings.

Floor Construction Detail

The floor slabs are constructed from precast concrete gothic beams which have a non-structural 2 inch screed over the top. The gothic beams are of hollow construction with 10mm ($^3/_8$") ribbed bars @ 110mm c/c located at the bottom with ½ inch cover typically.

In the four flats surveyed it was found that there was horizontal tie bars inserted into an average of every other grouted trough between the concrete beams at approximately 330mm c/c differing from the intent show on the existing drawing information. This was found to be inconsistent across the building with some areas having ties at in every trough, but others with ties missing for 5 troughs in a row. In the event of an internal gas explosion, it is likely that the tie bars would act sufficiently to resist disproportionate collapse, providing a tie force of 38.2kN/bar helping to resist a load of 20kN/bar based on a tie force requirement of 60kN/m. However, because of the inconsistency of installation found across the building, reliance on these tie bars cannot be made. Intrusive investigations revealed that in certain areas of John Cozens, no tie bars were located.

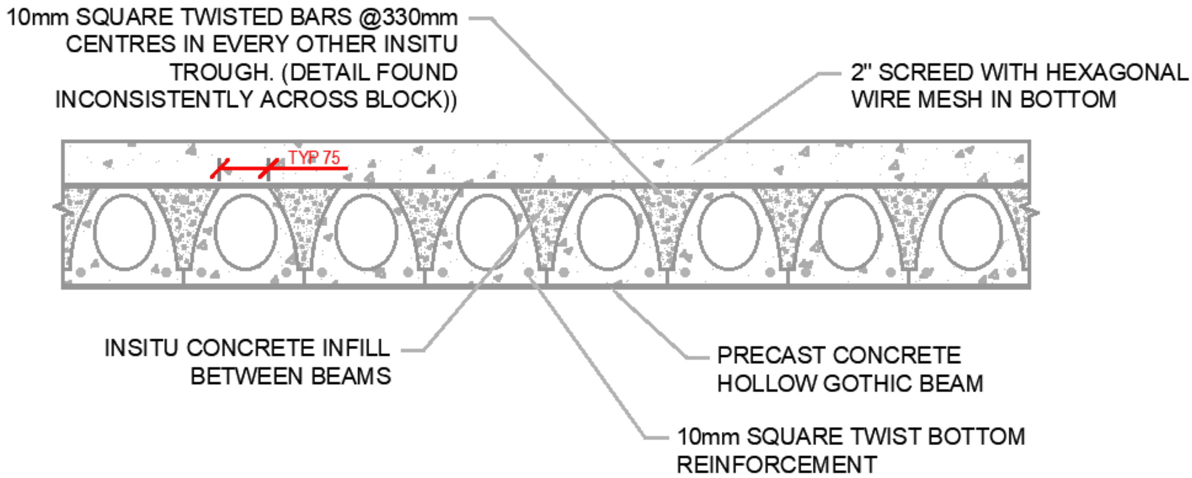


Figure 24 – Typical Gothic beam floor to wall joint construction detail.

4.5. Historic Strengthening

Following Ronan Point, John Cuzons had additional strengthening works completed with a combination of steel beams and columns to strengthen the floor slabs. Figure 25 shows the floor plan in flat 29 with the indicative locations of the strengthening works highlighted. It can be seen from the dimensions taken on site that the installed works have divided the span of the floor slab in half (approximately). Figure 25 and Figure 26 shows the details observed of the remedial strengthening works.

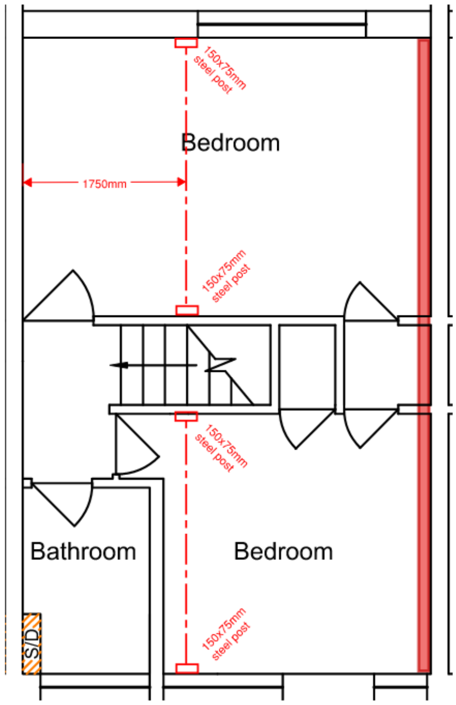


Figure 25 – Flat 18 floor plan showing location of strengthening beams and columns

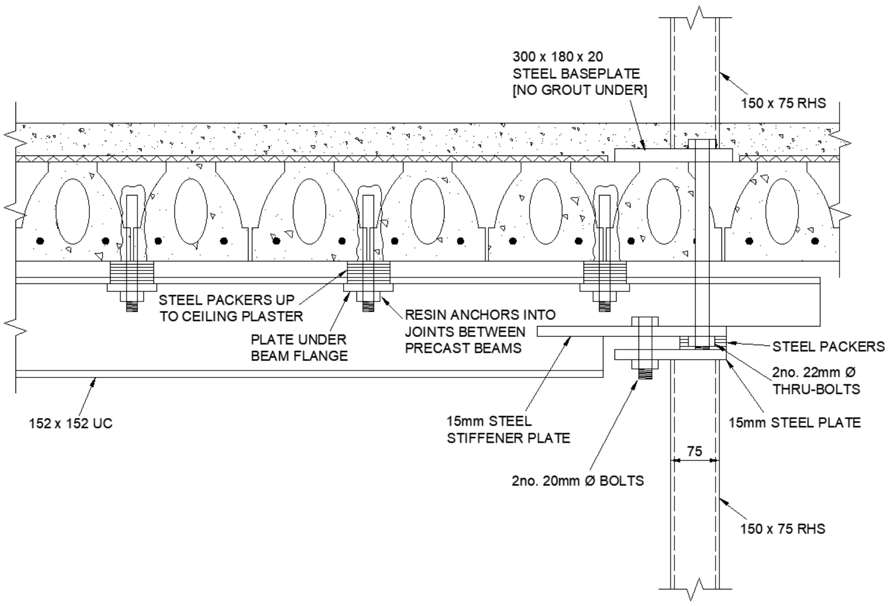


Figure 26 – Typical Historic strengthening works to floor slabs.

The remedial strengthening is fixed into the sofit of the existing slab at regular centres, these fixings help the slab when exposed to the overburden pressure in an uplift load case. Each beam has 16 fixing, 8 per side at 330mm c/c. Based on the total number of fixings, it was calculated that the fixings will need a capacity of circa 3.5kN. Additional pull load tests were completed on the fixings to confirm the capacity, the results are shown in section 5.6.

Facade Wall

The external front and back walls that infill between the structural cross walls are a double skin blockwork wall constructed from 75mm light weight blockwork with window and door openings inset. The masonry walls were noted in many instances to not be properly tied across the cavity and tied to the structural cross walls.

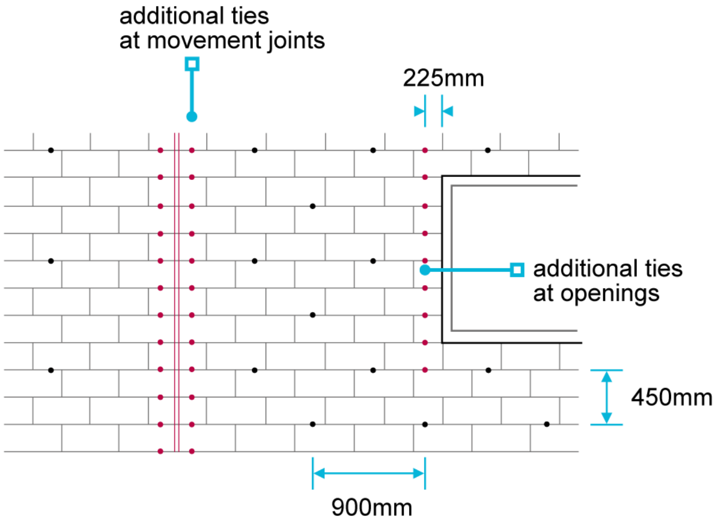


Figure 27 - Typical Wall tie detail (Ref NHBC Standard 6.1.18)

As indicated in Figure 27 wall ties across a cavity should generally be provided at 450mm vertical centres and 900mm horizontal centres, with additional ties provided around openings. In most instances one or two ties were found in the external wall panels and no additional wall ties were found around window and door openings. Additionally, there were limited ties between each masonry skin and the structural cross walls. It was noted that the cross walls had cast in wall ties, however most cases these had not been used when building the walls. The instructive investigations only allowed this to be reviewed in the inner skin of the cavity wall and therefore assessment of the outer skin cannot be made.

Figure 28 - Removed blockwork showing missing or unused wall tie

5. STRUCTURAL FIRE ASSESSMENT

5.1. Concrete Cover / Fire Resistance

The load bearing resistance of John Cozens House under approved document B table A2 is required to be 90 minutes.

The cover depths identified through the opening up works found the following:

- Floor Soffit Cover – 10mm – 21mm from breaking out, and 24mm on average from ferro scans.
- Flank Wall Cover – 13mm - 45mm, 28mm on average.
- Cross Wall Cover – 15mm - 40mm, 28mm on average, excluding shotcrete type screed.

The more conservative values have been used in our assessments. For the soffit, the 10-11mm results are generally closer to the design value of 12.5mm in the record drawings.

The cover depths identified, are generally in line with the relevant code at the time of construction, however, are relatively shallow by today's standards. The elements that are found to have below average cover will naturally provide less fire resistance. Furthermore, there will be variations of cover thickness within a single element.

For walls, the average cover is generally satisfactory for 90min fire resistance as per the table below, extracted from Eurocode 2, however, this cover is not universally present.

Table 5.4 - Minimum dimensions and axis distances for load-bearing concrete walls

Standard fire resistance	Minimum dimensions (mm) Wall thickness/axis distance for			
	$\mu_{fi} = 0{,}35$		$\mu_{fi} = 0{,}7$	
	wall exposed on one side	wall exposed on two sides	wall exposed on one side	wall exposed on two sides
1	2	3	4	5
REI 30	100/10*	120/10*	120/10*	120/10*
REI 60	110/10*	120/10*	130/10*	140/10*
REI 90	120/20*	140/10*	140/25	170/25
REI 120	150/25	160/25	160/35	220/35
REI 180	180/40	200/45	210/50	270/55
REI 240	230/55	250/55	270/60	350/60

* Normally the cover required by EN 1992-1-1 will control.

Note: For the definition of μ_{fi} see 5.3.2 (3).

Figure 29 - Extract from BS EN 1992-1-2 2004 - Eurocode 2 Design of Concrete Structures - Part 1-2 General Rules Structural Fire Design – Table 5.4

With an assumed floor soffit cover of 12.5mm, fire checks were conducted on the slabs to ensure they meet the required 90-minute fire resistance period for a stay put strategy. The slabs were found to pass in shear but fail in bending over their full span. By iterative calculations it is estimated that, for their full span, the slabs may be able to provide an estimated R60 fire resistance period which is in line with the code of the time however this is not in line with the time frame needed for a stay put strategy of 90 minutes, therefore additional measures are needed to increase the R fire resistance period to 90 minutes, please refer to the below decision tree.

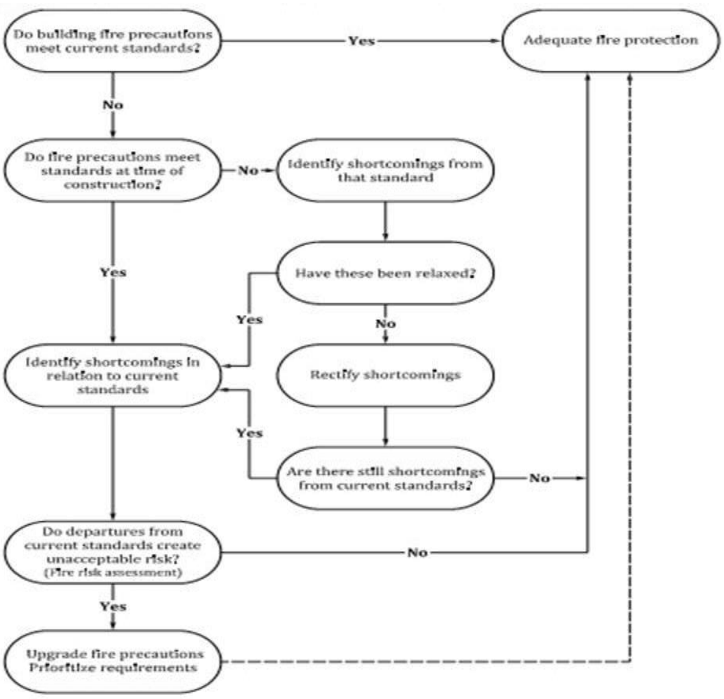


Figure 30 – Decision tree in relation to the requirement to upgrade fire precautions

The slabs appear to be satisfactory in both shear and bending if the 1970 retrofit steel frames, which reduce the slab spans, are taken into consideration. Therefore, in this scenario, the slabs may be able to provide R90 fire resistance period, assuming the steel frames themselves are adequately fire rated to the same level.

Additionally, the use of sprinklers could be used to reduce the room temperature in the event of a fire to an acceptable level to prevent failure of the reinforcement slab above for the required R90 time period.

It is worth noting, the Eurocode 2 guidance, that informs these calculations, is for solid slabs of minimum 200mm thickness. By contrast, in John Cozens the slabs are 171mm thick with internal cavities, the difference in construction will create a difference in how the heat dissipates within the slab. However, given both the known low cover and the solid nature of the slab to reinforcement the critical temperature calculation is considered adequate for the purposes of this report to inform the need for additional measures.

Further detailed analysis in the form of FEA could be undertaken should it be needed to inform the fire design further, please also refer to the fire engineers report and strategy in relation to this element of the assessment.

6. CONCRETE COVER AND TESTING

6.1. 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.

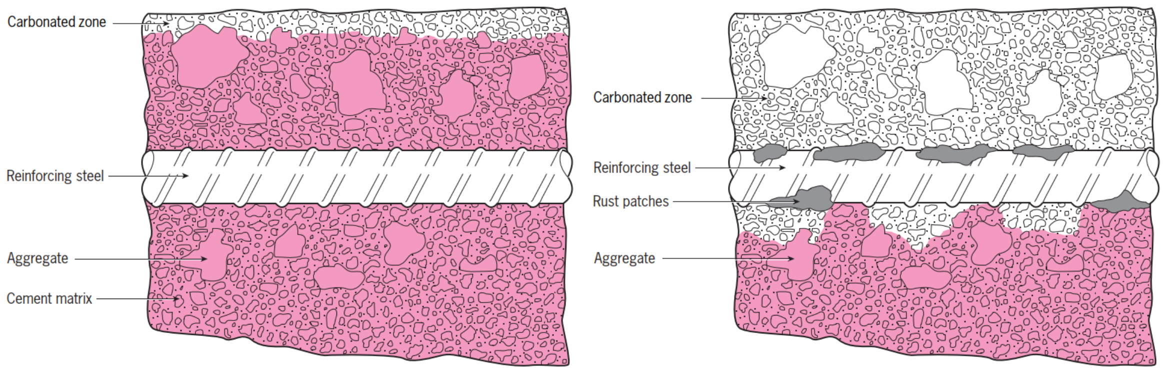


Figure 31 – (Left) Diagrammatic view of steel protected from carbonation-induced corrosion in partially carbonated concrete, (Right) Diagrammatic view of steel corroding in carbonated concrete.

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 3 below. The concrete testing has shown that the levels of carbonation have exceeded the depths of reinforcement and therefore the structure has lost the alkaline protection afforded by the concrete and is at high risk of deterioration through corrosion of the steel reinforcement.

Consequently, the structure is at significant risk of spalling in exposed environments with elements likely to break away from the main frame and drop to the floor from height leading to a high degree of risk of injury to the general public.

Table

Table 3 - Carbonation Depths

TEST REFERENCE	TEST LOCATION	MEMBER TYPE	CARBONATION DEPTH	MIN. COVER TO BAR	CARBONATION SURPASSED REINFORCEMENT?
DS1	Flat 43	Wall	5-10mm	25-30mm	No
DS2	Flat 43	Wall	5-10mm	25-30mm	No
DS3	Flat 43	Wall	50-55mm	25-30mm	Yes
DS4	Flat 43	Slab	5-10mm	12.5mm	No
DS5	Flat 43	Slab	0-5mm	12.5mm	No
DS6	Flat 43	Wall	5-10mm	25-30mm	No
DS7	Flat 43	Wall	5-10mm	25-30mm	No
DS8	Flat 43	Wall	60-65mm	25-30mm	Yes
DS9	Flat 17	Wall	55-60mm	25-30mm	Yes
DS10	Flat 17	Wall	50-55mm	25-30mm	Yes
DS11	Flat 18	Wall	50-55mm	25-30mm	Yes
DS12	Flat 18	Wall	60-65mm	25-30mm	Yes
DS13	Flat 18	Wall	35-40mm	25-30mm	Yes
DS14	Flat 33	Wall	20-25mm	25-30mm	Yes
DS15	Flat 33	Wall	20-25mm	25-30mm	Yes

DS16	Flat 33	Wall	20-25mm	25-30mm	Yes
DS17	Flat 33	Wall	30-35mm	25-30mm	Yes
DS18	Flat 17	Slab	5-10mm	12.5mm	No
DS19	Flat 17	Slab	5-10mm	12.5mm	No
DS20	Flat 18	Slab	0-5mm	12.5mm	No
DS21	Flat 18	Slab	0-5mm	12.5mm	No
DS22	Flat 33	Slab	0-5mm	12.5mm	No
DS23	Flat 33	Slab	0-5mm	12.5mm	No
DS25	Flat 17	Slab	5-10mm	12.5mm	No
DS26	Flat 18	Slab	0-5mm	12.5mm	No
DS27	Flat 33	Slab	70-75mm	12.5mm	Yes
DS28	Flat 33	Slab	5-10mm	12.5mm	No
DS9 FF	Flat 17	Wall	40-45mm	25-30mm	Yes
DS10 FF	Flat 17	Wall	70-75mm	25-30mm	Yes
DS11 FF	Flat 18	Wall	20-25mm	25-30mm	Yes
DS12 FF	Flat 18	Wall	70-75mm	25-30mm	Yes
DS13 FF	Flat 18	Wall	65-70mm	25-30mm	Yes
DS14 FF	Flat 3	Wall	50-55mm	25-30mm	Yes
DS15 FF	Flat 33	Wall	0-5mm	25-30mm	No
DS16 FF	Flat 33	Wall	0-5mm	25-30mm	No
DS17 FF	Flat 33	Wall	0-5mm	25-30mm	No

Notes:

- Some bars in this slab were exposed during the opening up and were shown to have surface corrosion. It is not clear whether this is due to the carbonated concrete no longer affording the steel protection (active corrosion), or inadequate storage of the bars prior to manufacture (historic corrosion).

- A shotcrete concrete covering has been provided to several walls, likely in an attempt to remediate the low concrete cover in these areas. This concrete contains a 2.5mm wire mesh at 100mm vertical and horizontal centres and will provide the reinforcement within the wall behind some protection by acting as a barrier.

- All carbonation tests have been considered against average cover depths observed within the structure.

In over half of the of the members tested, the carbonation depth was observed to have surpassed the depth of the embedded reinforcement. In these locations, the rebar is no longer within a passive environment and may therefore no longer have sufficient protection from the concrete to prevent corrosion. The carbonation depths observed did not appear to be excessive for a structure of this age, so it likely the main issue is the low concrete cover in some areas. It was noted that the reinforcement did not have consistent cover, even within a single wall panel, suggesting there was poor quality control during construction.

6.2. Chlorides

Chloride testing was carried out by drilling the concrete with a hammer drill and the dust created collected and transferred into sealable bags. 69 dust samples were collected from across the flats tested. The concrete sampling was carried out in accordance with BRE IP 21/86. In-situ depth of carbonation testing was carried out to BS EN 14630: 2006. The chloride content and cement content tests were carried out to BS 1881: Part 124:2015. The compressive strengths of the cores were carried out to BS EN 12504-1:2019 and pull-out tests were carried out in accordance with BS 8539.

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 completed in the 1970s, meaning the property is circa 50 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 31.

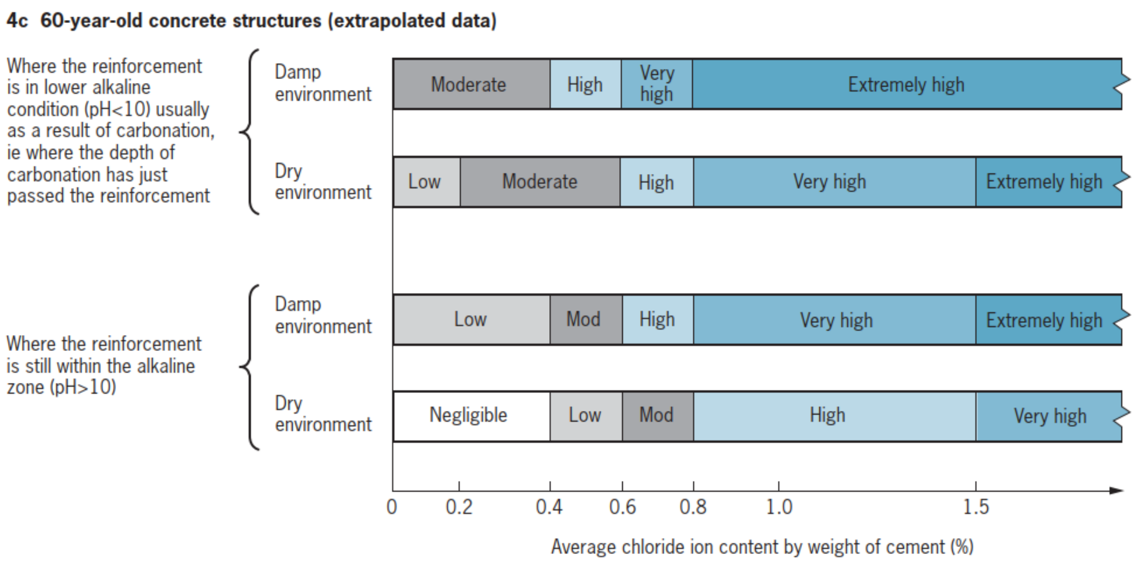


Figure 32 – Estimated risk of corrosion associated with carbonation, chloride content and environment.

The testing data has been assessed based on the BRE guidance to create table 4, showing the risk of steel reinforcement corrosion in each of the areas tested.

Table 4 - Interpretation of Chloride Content Testing with BRE Digest 444 Part 1
All samples are considered to be taken from within a 'dry' atmosphere.

TEST REFERENCE AND LOCATION		MEMBER TYPE	CARBONATION REACHED / SURPASSED REINFORCEMENT	CHLORIDE CL % BY MASS OF CEMENT	RISK OF STEEL REINFORCEMENT CORROSION (BRE DIGEST 444 PT1)
DS1	Flat 43	Wall	No	0.24	Negligible
DS2	Flat 43	Wall	No	0.18	Negligible
DS3	Flat 43	Wall	Yes	0.18	Low
DS4	Flat 43	Slab	No	0.18	Negligible
DS5	Flat 43	Slab	No	0.20	Negligible
DS6	Flat 43	Wall	No	0.22	Negligible
DS7	Flat 43	Wall	No	0.22	Negligible
DS8	Flat 43	Wall	Yes	0.80	High
DS9	Flat 17	Wall	Yes	0.12	Low
DS10	Flat 17	Wall	Yes	0.18	Low
DS11	Flat 18	Wall	Yes	0.14	Low
DS12	Flat 18	Wall	Yes	0.16	Low
DS13	Flat 18	Wall	Yes	0.16	Low
DS14	Flat 33	Wall	Yes	0.06	Low
DS15	Flat 33	Wall	Yes	0.08	Low
DS16	Flat 33	Wall	Yes	0.16	Low
DS17	Flat 33	Wall	Yes	0.26	Moderate
DS18	Flat 17	Slab	No	0.22	Negligible
DS19	Flat 17	Slab	No	0.18	Negligible
DS20	Flat 18	Slab	No	0.18	Negligible
DS21	Flat 18	Slab	No	0.18	Negligible
DS22	Flat 33	Slab	No	0.20	Negligible
DS23	Flat 33	Slab	No	0.22	Negligible
DS25	Flat 17	Slab	No	0.18	Negligible

DS26	Flat 18	Slab	No	0.16	Negligible
DS27	Flat 33	Slab	Yes	0.14	Moderate
DS28	Flat 33	Slab	No	0.16	Negligible
DS9 FF	Flat 17	Wall	Yes	0.18	Moderate
DS10 FF	Flat 17	Wall	Yes	0.12	Low
DS11 FF	Flat 18	Wall	Yes	0.08	Low
DS12 FF	Flat 18	Wall	Yes	0.08	Low
DS13 FF	Flat 18	Wall	Yes	0.32	Moderate
DS14 FF	Flat 3	Wall	Yes	0.18	Low
DS15 FF	Flat 33	Wall	No	0.32	Moderate
DS16 FF	Flat 33	Wall	No	0.14	Low
DS17 FF	Flat 33	Wall	No	0.14	Low

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

- The chloride contents in the concrete tested was found to be generally low, in most locations, with the exception of a few tests.

- Concrete members tested were generally found to have a negligible or low risk of corrosion to the embedded reinforcement, with several tests indicating a moderate risk. As stated in the previous subsection, it appears the elements which are at the 'low' and 'moderate' risk of corrosion are so mainly due to the low concrete cover to the rebar.

- One location was indicated to have a 'high' risk of corrosion to embedded reinforcement. This elevated risk of reinforcement corrosion maybe due to the increased chlorides identified through laboratory testing. However, when considering the wider structure, it is perceived that this is an isolated location of elevated chlorides and it not indicative of the structure as a whole.

- During the opening up works some degree of corrosion (typically minor surface corrosion) was noted to the reinforcement. It is not clear whether this is due to the carbonated concrete no longer affording the steel protection (active corrosion), or inadequate storage of the bars prior to manufacture (historic corrosion)

6.3. Cement Composition

The structural performance of concrete is 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 to determine the chloride content can be seen in Table 5. 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 6. 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 7.

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 ($3CaO \cdot SiO_2$ and $CaO \cdot SiO_2$), 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.*'

Table 5 - Cement composition test results

TEST MEMBER	SIO2	CAO	TOTAL CEMENT CONTENT
Wall (Precast)	2.5	30.2	37.2
Wall (In-situ)	2.1	31.6	36.1
Slab	2.8	25.4	41.1

Table 6 - Interpretation of cement composition testing

TEST MEMBER	SIO2	CAO	TOTAL (SIO2 + CAO)
Wall	6.7%	81.2%	87.9%
Wall	5.8%	87.5%	93.3%
Slab	6.8%	61.8%	68.6%

Table 7 - Comparison of interpreted results with BS EN 197-1:2011

TEST MEMBER	CEMENT CONSISTS OF AT LEAST 2/3 (CAO + SIO2)	THE RATIO OF CAO / SIO2 > 2.0
Wall	87.9 > 66.6 ∴ PASS	12.1 > 2.0 ∴ PASS
Wall	93.3 > 66.6 ∴ PASS	15.1 > 2.0 ∴ PASS
Slab	68.6 > 66.6 ∴ PASS	9.1 > 2.0 ∴ PASS

By inspection of the interpreted results, the quantities of Silica (SiO_2) and Calcium Oxide (CaO) satisfy the expected proportions for today's standards. The concrete is therefore likely to be offering adequate protection for embedded reinforcement.

6.4. Compressive Strength

In order to assess the robustness of the concrete elements forming John Cozens, 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 John Cozens were from cross walls and flank walls. It was not possible to undertake core sampling of floor slabs as the floors were constructed from precast, hollow beams so a solid core sample would not have been retrieved, each 75mmmm in diameter. These were sent to the Perry Testing Ltd laboratory, and the compressive strength of each core determined. The results of the testing can be seen in Table 8.

Table 8 - Compressive strength results from the core samples taken in John Cozens

LOCATION	SAMPLE	CORE LOCATION	COMPRESSIVE STRENGTH
3rd Floor	C1	Cross Wall	37.3 N/mm^2
3rd Floor	C2	Cross Wall	29.0 N/mm^2
3rd Floor	C3	Cross Wall	30.1 N/mm^2
3rd Floor	C4	Cross Wall	28.6 N/mm^2
3rd Floor	C5	Flank Wall	33.8 N/mm^2
3rd Floor	C6	Flank Wall	21.5 N/mm^2
3rd Floor	C7	Flank Wall	24.5 N/mm^2
3rd Floor	C8	Flank Wall	19.9 N/mm^2
3rd Floor	C9	Cross Wall	29.3 N/mm^2
5th Floor	C10	Cross Wall	28.0 N/mm^2
5th Floor	C11	Cross Wall	37.6 N/mm^2
5th Floor	C12	Cross Wall	22.4 N/mm^2

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 13791:2019 – '*Assessment of in-situ concrete strength in structures and precast concrete components*' and Concrete Advice No.68 – *'Assessment of in-situ concrete strength using data obtained from core testing.'*

The calculations, based on the core results, show that the characteristic compressive strength of the concrete walls at John Cozens is:

Table 9 - Compressive strength results for John Cozens

TEST LOCATION	COMPRESSIVE STRENGTH
Cross Wall	**21.1N/mm^2**
Flank Wall	**19.9N/mm^2**

The above value for the Cross walls is based on a standard deviation with the highest and lowest values removed, due to limited number of samples obtained the flank wall compressive strength is taken as the lowest test value achieved. It may be an improved compressive strength could be determined through further core sampling.

6.5. Pull tests

To utilise the post installed steel frame strengthening the resin anchor fixings into the sofit of the slabs needs to be confirmed as exceeding 3.5kN. Therefore, a pull test was completed on a range of fixings across John Cozens. The results can be seen in Table 10 below. All fixings reached the 5kN test load for a duration of 3 minutes. With the capacities achieve it is expected that the steel frames could be engaged and utilised to strengthen the slabs.

Table 10 - Pull test results for John Cozens

LOCATION REFERENCE	LOAD (KN)	LOAD DURATION (MIN)	NOTES
1 – DS4	5	3	No Observed Movement
2 – DS5	5	3	No Observed Movement
3 – DS22	5	3	No Observed Movement
4 – DS23	5	3	No Observed Movement
5 – DS8	5	3	No Observed Movement
6 – DS20	5	3	No Observed Movement
7 – DS21	5	3	No Observed Movement
8 – DS21/1	5	3	No Observed Movement
9 – DS18	5	3	No Observed Movement
10 – DS19	5	3	No Observed Movement

7. JOHN COZENS STRUCTURAL ASSESSMENT

7.1. Assessment Criteria

The John Cozens 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 block is not currently fitted with a piped-gas supply, and as such the main structural members do not need to be assessed for the enhanced overpressure of 34kN/m^2.

  - The structure shall, instead, be assessed against the reduced overpressure of 17kN/m^2 – this is the value associated to a block without a piped-gas supply, but could be subjected to an explosion from sources such as aerosols or LPG canisters etc.

- 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.

7.2. 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 block. Based on the definitions provided by Building Regulations Approved Document A the block falls into Consequence Classes 2b. The block therefore requires effective horizontal and vertical ties. The details for the joints between floors and walls can be seen in Section 4.4.

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:
<u>Horizontal Ties:</u> Insufficient due to inconsistencies
<u>Vertical Ties:</u> Insufficient

Ties should have been installed in every trough. Generally, ties were located at every other or third trough with some missing entirely. As the tie provision is not consistent in all areas tests it is assumed that horizontal ties are ineffective through lack of provision. Through analysis it has been concluded that the existing provision of horizontal ties does meet the requirements for the resisting the minimum tie force but the inconsistency in the provision of the horizontal ties means they cannot be relied on.

There is a lack of continuous ties between the precast panels, **therefore the joint is considered to not have adequate provision of vertical ties.**

The cross-wall joint is therefore **<u>insufficient</u>** to pass the assessment for a Consequence Class 2b building.

Flank Wall / Floor Slab Joints (Precast)

The assessment of the cross wall / floor slab joint has shown that:
<u>Horizontal Ties:</u> Insufficient
<u>Vertical Ties:</u> Insufficient

Flank wall to floor ties are installed at circa 330mm c/c or are not present. From investigations it was found that the bars were tied to a 12mm lacer bar in the wall, which would provide a tie. The existing provision of horizontal ties does meet the requirements of the resisting the minimum tie force but the inconsistency in the provision of the horizontal ties means they cannot be relied on.

There is no continuous tie present between the top of one panel and the base of the that above, **therefore the joint is considered to not have an adequate provision of vertical ties.**

The precast flank wall joint is therefore **<u>insufficient</u>** to pass the assessment for Consequence Class 2b.

Flank Wall / Floor Slab Joints (Insitu)

The assessment of the cross wall / floor slab joint has shown that:
<u>Horizontal Ties:</u> Partial tie
<u>Vertical Ties:</u> Sufficient

The In-situ walls have 10mm square twisted bars cast into the walls at circa 300mm c/c. **The existing provision of horizontal ties does meet the requirements of the resisting the minimum tie force.**

Continuous vertical reinforcement was found with the 6mm reinforcement lapped between storey heights.

The insitu flank wall joint is therefore **sufficient** to pass the assessment for Consequence Class 2b when considered with the strengthening frame.

Cross Wall Vertical Joints

The vertical joints between abutting wall panels are 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 19 below, to prevent the joint from uncontrolled separation.

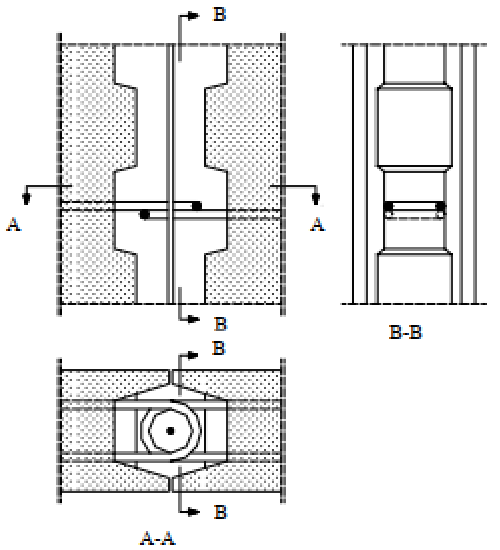


Figure 33 - Typical shear key connection at a vertical joint between wall panels

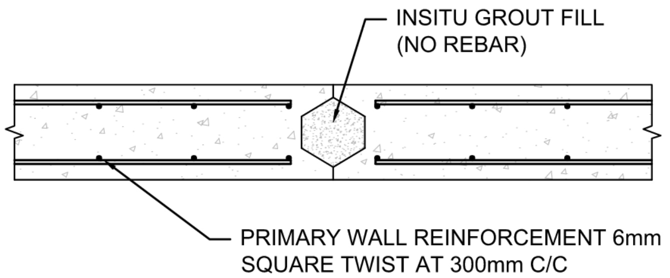


Figure 34 - Joint detail between cross wall panels as identified on site.

However, within John Cozens, no vertical lacer bar or link bars were located between abutting wall panels. Therefore, **the joint does not possess an adequate horizontal tie** between abutting wall panels.

Table 11 – John Cozens tie details summary table

John Cozens (Consequence Class 2B)			
Joint Type	**Adequate horizontal tie**	**Adequate Vertical Tie**	**Notes**
Flank Wall (Precast)	X	X	Insufficient
Flank Wall (In-situ)	X	a	Insufficient
Cross Wall	X	X	Insufficient
Wall to Wall Joints	X	X	Insufficient*

John Cozens – LPS Criterion 1 – Adequate Provision of ties
Insufficient provision of ties

* The tie in wall-to-wall joints was non-existent. In the absence of any ties, the performance of the tie between these panels is considered to be inadequate.

7.3. 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 were carried out for the block, based on the findings of the intrusive investigations carried out on each of the main loadbearing members.

The calculations are carried out to Eurocode, the assessment was carried out using an overpressure of 17kN/m^2 to comply with the regulations for accidental loading for a building without a piped-gas supply. The calculations show that the structural elements that form John Cozens, with the exception of flank walls levels 1, 2 and 3, and cross walls at levels 1 and 2, are insufficient to resist a loading of this magnitude.

The following table summarises the findings:

Table 12 – John Cozens robustness assessment summary table

STRUCTURAL ELEMENT	JOHN COZENS 17KN/M2 OVERPRESSURE (NO PIPED-GAS SUPPLY)	NOTES
Floor Slab (Downward)	X	Inadequately Robust
Floor Slab (Uplift)	X	Inadequately Robust
Floor Slab Reduced Span (Downward)	a	Adequately Robust
Floor Slab Reduced Span (Uplift)	X	Inadequately Robust
Flank Wall Level [5-9]	X	Inadequately Robust
Flank Wall Level [1-4]	a	Adequately Robust
Cross Wall Level [4-9]	X	Inadequately Robust
Cross Wall Level [1-3]	a	Adequately Robust

Wall panels are adequately robust below the 3rd floor where the axial load imposed by the weight of the building provides a favourable scenario against failure. At levels 4-9 the wall panels fail under the 17kN/m² overpressure loading.

John Cozens – LPS Criterion 2 – Adequate Collapse Resistance
Unsatisfactory - Inadequate Collapse Resistance

7.4. LPS Criterion 3 – Alternative Load Paths

For a block to satisfy Criterion 3 the structure must be able to mobilise alternative load paths in the event of an explosion. In the event of an explosion without a piped-gas supply, the bounding enclosure area would be considered to be a single room within the flat. The overpressure from such an event is considered to act on all elements within this bounding enclosure simultaneously.

In an LPS frame typically every element is considered to be critical. The connections between elements can be, at best, considered to be acting as flexible joints (with unconfirmed stiffness), rather than a true fixed connection. In this scenario, failure of any element in the system is likely to cause a mechanism and disproportionate collapse.

John Cozens – LPS Criterion 3 – Alternative Load Paths
Unsatisfactory - Unable to Mobilise Alternative Load Paths

The inability to mobilise alternative load paths combined with the anticipated failure of elements under accidental loading conditions could result in any one of the below mechanisms occurring.

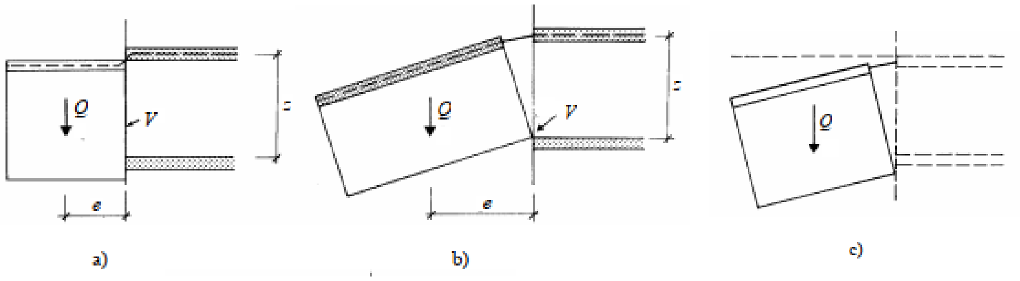


Figure 35 - Examples of collapse mechanisms for a cantilevering wall above a damaged area, a) joint slip mechanism, b) rotation mechanism, c) combined slip-rotation (fib, 2008)

7.5. Summary of LPS Criteria Checks

John Cozens has been assessed in its current condition against the three LPS Criteria. The assessment has shown the block fails all three of the checks and is therefore inadequately robust to resist disproportionate collapse.

	John Cozens	
LPS CRITERION	**PASS / FAIL**	**NOTES**
LPS 1	X	Inadequately Tied
LPS 2	X	Inadequately Robust
LPS 3	X	Inadequate Mobilisation of Alternative Load Paths
	John Cozens – Conclusion	
	Insufficient	

8. CONCLUSION

8.1. Key Findings Summary

A summary of key findings is included below:

Visual Assessment Findings

- Spalling of concrete has been noted in several locations around the building, including the individual balconies on the back of the building.
- Poor quality construction of gothic beams has been observed in several locations. With poor concrete compaction and positioning of reinforcement.
- Corrosion of the primary balustrade along the access walkways is widespread with temporary scaffold support frames previously installed having been used to protect the worst areas.

Concrete Testing Findings

- It was found that the majority of tests gave a negligible or low risk of corrosion to the embedded reinforcement, however, several tests indicating a moderate or high risk. As stated in previous subsections, it appears the elements which are at the 'low' and 'moderate' risk of corrosion are so mainly due to the low concrete cover to the rebar and levels of carbonation.
- The precast cross walls and flank walls have a characteristic compressive strength of 21.1N/mm^2, and 19.9N/mm^2, respectively. [Note this is based on core sampling of the wall elements only. The floor slabs could not be cored due to occupied flats above/below.]
- Concrete cover, although believed to be typical of the period of construction may not offer sufficient resistance in the event of a fire for stay put or for the purposes of prevention of reinforcement corrosion and consequently lead to spalling concrete.
- Pull tests completed on the strengthening frame connection to the concrete floor slab indicate that the fixings can achieve 5kN, allowing them to be utilised in an accidental load case scenario.

Intrusive Investigation Findings

- No effective vertical ties were located between precast concrete wall panels.
- Horizontal ties between cross walls and flank walls observed to be inconsistently installed, with an adequate tie being provided in some instances but not regularly across the building.
- Cast in L-bars provide a horizontal tie into the in-situ flank walls. The L-bars extend 450mm down into the flank walls and 900mm into the floor slabs.
- Precast cross walls have a 22mm dowel and levelling plate located 19 inches (485mm) from panel joints to provide adjustment during installation. These dowel pockets have not been grouted once panels are level.
- No lateral tie has been found at precast panel joints with joints being grouted only.
- A 50mm shotcrete remedial detail has been applied to various walls across the building where blockwork linings are not present, containing a 2.5mm wire mesh at 100mm c/c.
- A limited number of masonry ties have been found between the two skins of masonry forming the outer wall of the building, with very few ties found in use between the walls and the structural cross walls.

8.2. Conclusion

The following conclusion is drawn from the results of investigations completed in accordance with the structural assessment procedure outlined in section 4. The outcome of the assessment is that John Cozens in its current state is inadequate to resist disproportionate collapse due to inadequate provision of ties between elements and failure of some precast elements under accidental loading.

It is also assessed that the structure would not be able to mobilise alternative load paths effectively in the event that a key element was removed through accidental loading due to the inadequate tie reinforcement provision.

In some locations, tie reinforcement was not observed in the same regularity expected and will not respond adequately under load. Due to the frequency this was observed it is considered that this may occur multiple times throughout the structure which is likely to compromise the global structure in an accidental loading event.

Based on average cover slabs have been shown to pass in shear but fail in bending over their full span based on an R90 fire requirement. For the classification of building a minimum of R90 is required. It has been shown that by utilising the existing remedial steel frames an R90 fire period may be achieved, provided the steel frame have adequate protection to meet the same and following a review of the fire event by the fire engineer.

Alternatively, the use of sprinklers could be used to reduce the room temperature in the event of a fire to an acceptable level to prevent failure of the reinforced slab above for the required R90 time period.

9. RECOMMENDATIONS

The block primary structure appears to be in an adequate condition with the exception of certain elements, namely the balconies and balustrades.

However, under accidental loading the structure is assessed to be inadequate in the event of a non-piped gas explosion. Visual observations and non-intrusive investigations also identified factors which may result in the structure performing sub-optimally in the event of a fire. Works are recommended to address the failings of the accidental loading assessment to reduce the risk to as low as reasonably possible.

It is understood that historically, the piped gas supply was removed from the structure, which is an important preventative measure to reduce the risk of an explosion. This reduced risk of an explosion or accidental loading which could lead to a disproportionate collapse event should be further enhanced by adopting the following recommended measures:

Immediate Term (0-6 Months)

1. Continuation of the updated building evacuation strategy to a simultaneous evacuation, with the continued waking watch across St Jude's. This is a short-term measure in line with Government guidance (Evacuation guidelines for fire and rescue services (accessible))
2. Installation of fire detection and alarm system (BS5839 - 1 Cat L5) to replace waking watch in accordance with NFCC guidance
3. Regular inspections for and immediate ban on:
    a. Any gas cannister/bottles/cylinders being used or stored within the dwellings, along with a complete ban on any other potentially explosive substances (including high-capacity batteries which may be found in items including e-scooters/e-bikes and some newer models of mobility scooters).
    b. Portable gas cookers – viewed as high risk as they have the potential to be left on whilst unignited, causing a leak that may then be unintentionally ignited, causing an explosion and excessive pressures being applied on the structures.

   c. To limit hoarding to minimise fire loads in flats
4. Removal of gas supply to laundry rooms and presence of diesel generators near the building that could increase the risk of an accidental loading scenario.
5. Full condition survey of the balustrades around John Cozens, temporary support provided to those in a critical condition with a design and programme developed to replace all the balustrades.
6. Detailed condition surveys of the balconies and walkways due to carbonation of the concrete to identify deteriorated and degraded areas or the structure to enable repairs as necessary.
7. Erection of the non-combustible scaffold fan to the base of the block to prevent falling concrete.
8. Detailed wind analysis of the block to be undertaken to assess peak forces on the external masonry walls with remedial design / strengthening options.

Medium Term (6 months -2 Years)

1. Installation of sprinkler protection to BS 9251 Category 4 and conversion of existing detection system, or enhancement of the fire protection of the structure to increase the fire resistance.
2. Repairs to concrete on residential balconies and communal walkways and Removal of residential balconies.
3. Carry out an options appraisal to understand the cost benefit of upgrading the structure to resist disproportionate collapse then:
   a. Upgrade the structure through ties or strengthening to resist disproportionate collapse forces and provide a robust structure.
   b. If strengthening works are unviable re-assess the risk measures in place and determine any further measures that will enable the block to remain in service over a short term until decant can be undertaken for demolition.
4. Repairs and or replacement of the residential balconies due to deterioration from carbonation.
5. Remedial repairs to the escape walkways following detailed surveys.
6. Remedial repair works to the external masonry walls, or overclad the existing envelope.
7. If the block is to be retained investigate and assess the foundations for deterioration and chemical attack.

Long Term (3-5 years+) Continued Inspections

Considering the buildings type and height the following recommendations are made, which align with BRE recommendations:

1. A programme of visual inspections at intervals of 1 year, 2 years and 5 years following this initial appraisal, and then every 5 years subsequently to the external envelope (including parapets and balconies) to identify potential hazards from falling debris.

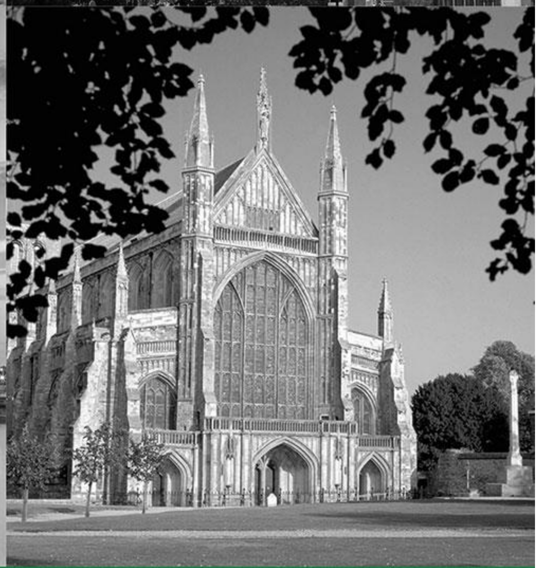
2. Visual inspections at 10-year intervals to structural joints which are vulnerable to water penetration; locations such as flank walls and roofs.

3. Full appraisal of the whole building at 20-year intervals

Should the risk reduction measures fail to effectively control the risk of disproportionate collapse to acceptable levels, and investment into strengthening works prove uneconomically viable, demolition of the block might be considered as a final long-term approach for the block. However, we would recommend that this decision should only be taken following the completion of a remedial strengthening design review, supported by the risk and cost benefit analyses recommended above to ensure that demolition is the best approach.





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