RIDGE

ST JUDES – CHARLETON HOUSE
STRUCTURAL ROBUSTNESS ASSESSMENT

BRISTOL CITY COUNCIL
December 2024

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STRUCTURAL ROBUSTNESS ASSESSMENT

December 2024

Prepared for

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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 majority of 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 end block of flats was found to be of a load bearing masonry construction with similar gothic beam floors spanning between the masonry. 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. As Charleton House shares the lift and stair core structure with Haviland House the assessment criteria in relation to fire and the requirements for this have been applied in this assessment given the potential to impact the escape route within the 90-minute period. The criteria and results relating to Charleton House are as follows:

Table 1 - Summary of LPS Criteria for Charleton House

LPS CRITERION	ASSESSMENT	COMMENTS
Criterion 1 – Precast Frame Adequate ties within joints	**Insufficient**	Charleton House 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 1 – Brickwork Flats Adequate ties within joints	**Insufficient**	Charleton House is a class 2B building that requires both vertical and horizontal ties. No horizontal ties were found between the gothic beam floor and load bearing masonry walls. Investigations also found no adequate vertical ties between wall panels.
Criterion 2 – Precast Frame Adequate strength to resist Accidental Loads	**Insufficient**	The floors cannot resist the overpressure requirement for non-piped gas supply of 17kPa. The wall panels cannot resist the overpressure requirement for non-piped gas supply of 17kPa.

Criterion 2 – Brickwork Flats Adequate strength to resist Accidental Loads	**Insufficient**	The floors cannot resist the overpressure requirement for non-piped gas supply of 17kPa. The wall panels above level 4 cannot resist the overpressure requirement for non-piped gas supply of 17kPa.
Criterion 3 – Precast Frame 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.
Criterion 3 – Brickwork Flats Ability to mobilise alternative load paths	**Sufficient**	The use of alternative load paths is considered to be feasible in the masonry portion of the building. A failure of a section of the masonry panels will lead to a redistribution of load through archiving of the masonry over the failure zone.

Table 2 - Assessment criteria summary for Charleton House

ASSESSMENT CRITERIA	ASSESSMENT	COMMENT
Fire Resistance	**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.
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 supports and tie details	**Insufficient**	The external masonry cladding on the building, consisting of two layers of blockwork, were found to be inadequately tied to the primary concrete walls and floors and also had inadequate ties between the two skins of masonry.
Balustrades Condition of metal balconies	**Insufficient**	Balustrades around the building, particularly along the shared access walkways, were noted to be severely corroded particularly at the base connection to the floor. The condition of the balustrades requires replacement of the full system.

In addition to the inspection and assessment of the concrete frame, visual surveys of the overall building condition were 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 wall ties both between the cavities and back to the structural frame. The access walkway balconies along the front of Charleton House were noted to have spalling concrete on the underside.

The building 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. To mitigate any long-term risks of the foundations degrading, further investigations of the footings could be completed.

Recommendations

Considering the above results of the assessment & the general condition of the block, our recommendations 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 Charleton House, 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. 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. Remedial repairs to the escape walkways following detailed surveys.
5. Remedial repair works to the external masonry wall, or overclad the existing envelope.
6. 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 proposed not effectively limit the residual 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 Charlton House, Haviland House, Langton House and John Cozens.

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 Charleton 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. Refer to Appendix A for the detailed testing results of Charleton House.

This report documents the main findings of the investigation and the findings of the subsequent structural assessment into the robustness of the Charleton House 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 7, 12, 14 & 20.

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.

There are two types of flat in Charleton House, the majority are 3-bedroom maisonettes, with the exception of five flats on the southern end of the block which are single bedroom apartments. The floors are constructed from a series of precast concrete gothic beams with an in-situ concrete screed over the top. It was therefore not possible to obtain core samples from floor slabs for compressive testing due to the nature of the construction.

2.5. Purpose of report

The purpose of the report is to advise on the cross-wall construction of Charleton House 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

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

The dwelling block, Charleton House 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 House and Langton Houses (Blocks A & B) being built as part of Stage 1 and Haviland House & John Cozen's being built as part of Stage 2.

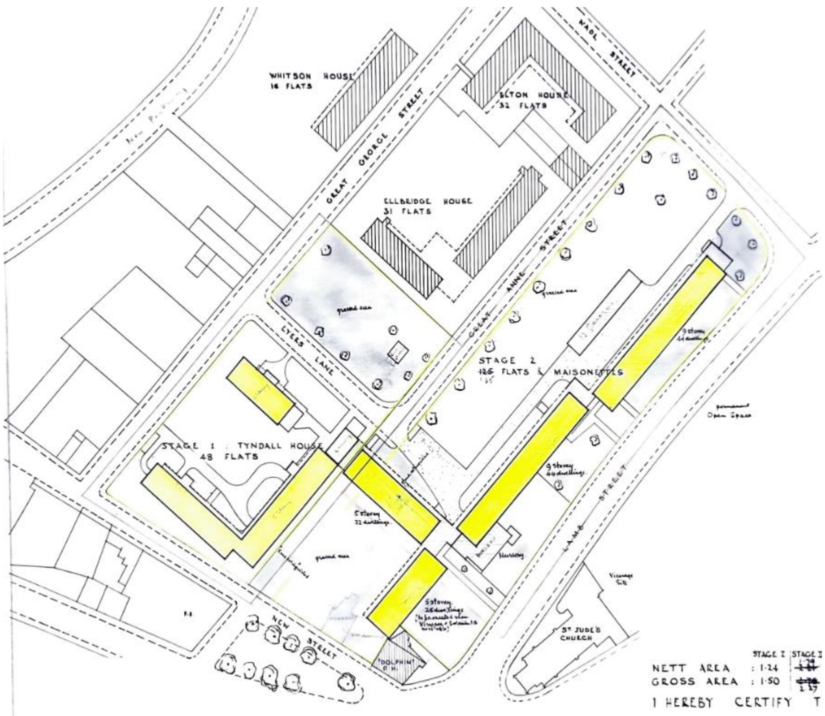


Figure 3 - As built site plan of Blocks A, B, C & D

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

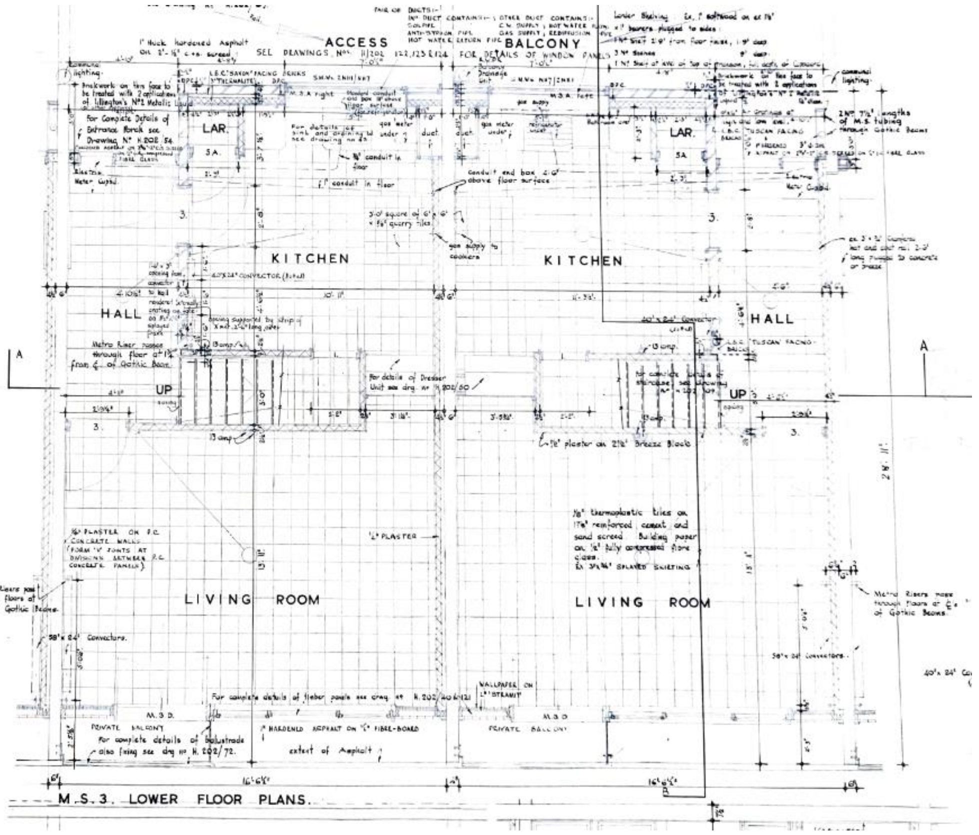


Figure 4 - **Charleton House** Lower floor record plans

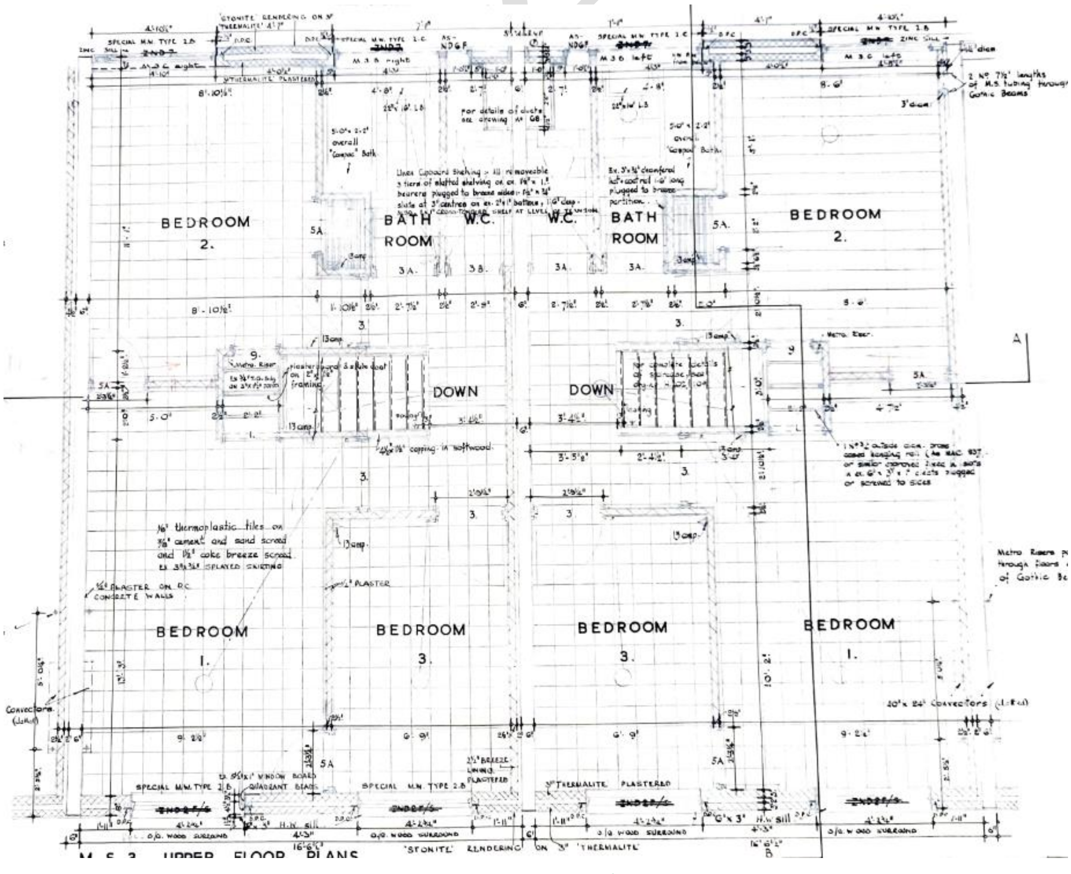


Figure 5 - **Charleton House** upper floor record drawings

Each flat in Charleton House 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 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 precast concrete gothic beam floors. The access walkways are accessed via a lift & stair core at either end of Charleton House where it connects with the adjacent blocks.

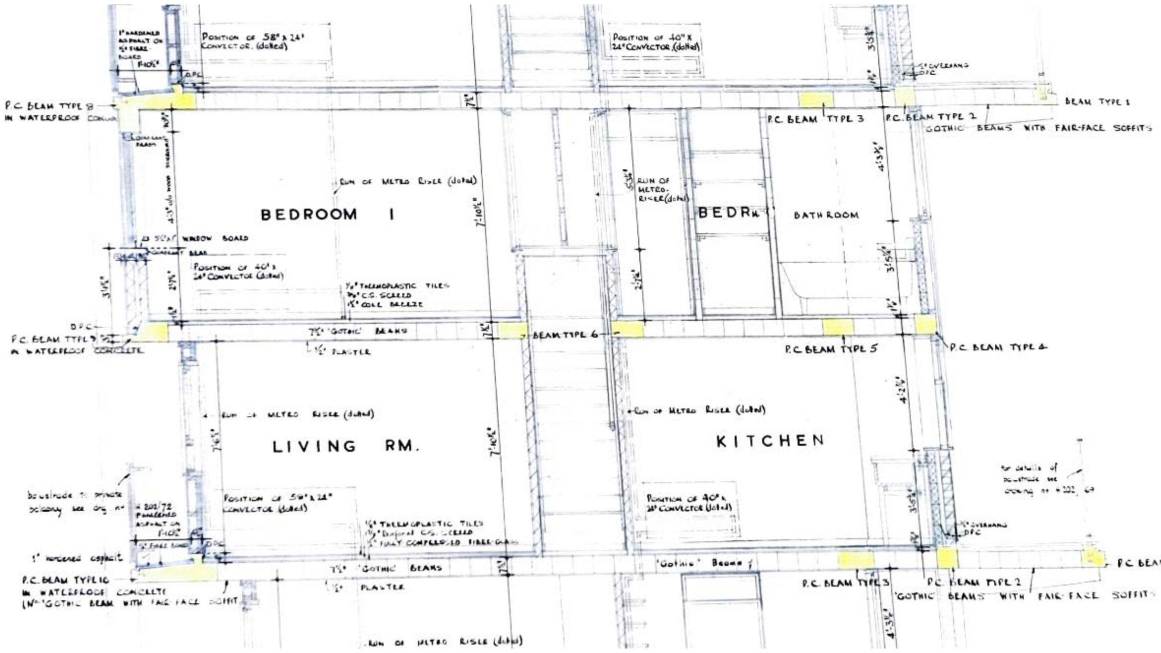


Figure 6 – Charleton House typical record drawings section.

Each flat has a balcony accessed from the lower-level living area extending approximately 1000mm off the rear elevation of the building. The balconies on Charleton House are formed by the external wall being stepped in from the external building line, with the floor formed from gothic beams spanning on to the precast cross walls, similar to the wider building construction.

The construction of the majority of Charleton House follows a similar precast cross wall and gothic beam floor arrangement, with the main difference being the construction of the single bedroom flats at the south end of the building. Flats 13, 14, 21 & 22 are located within this section of the building. This section of the building has a similar gothic beam floor but has load bearing masonry walls on all four sides supporting the floors and roof.

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 of the precast cross wall section of Charleton House follows the hierarchical approach adopted by BRE 511 as shown in Figure 7 below.

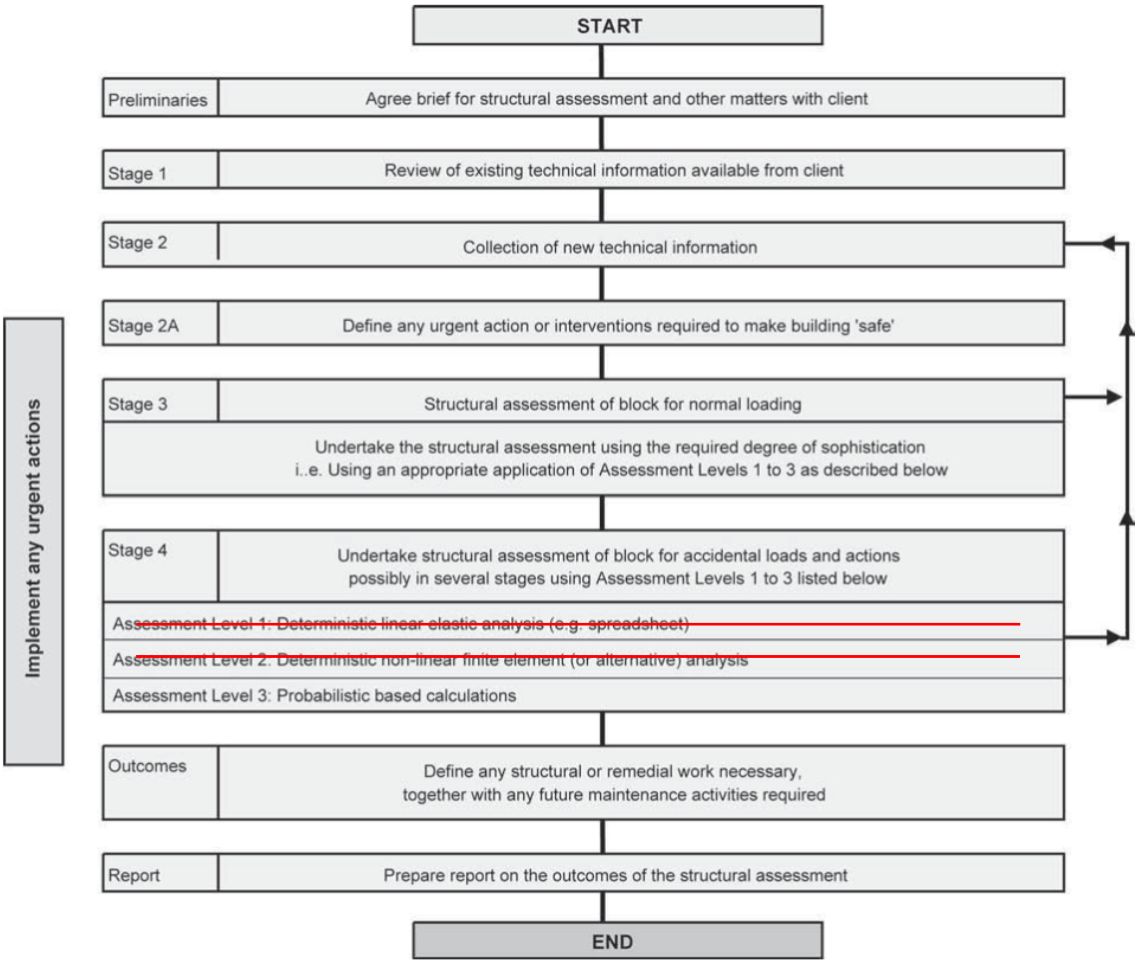


Figure 7 - Extract of BRE 511 figure 34 'Main steps in the structural assessment process'.

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 7, 12, 14 & 20, were subjected intrusive breaking out to confirm structural details, this gave a sample of 18% of the total number of flat in Charleton House, and 16.6% of the cross-wall construction flats.

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 7, 12, 14, 20 were subjected to the intrusive opening up works.

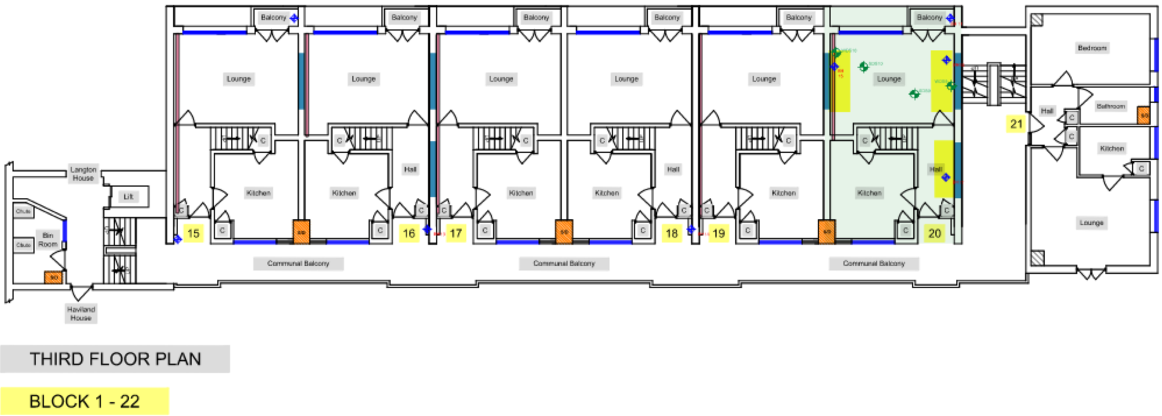


Figure 8 – Third floor plan for Charleton House showing the flats inspected.

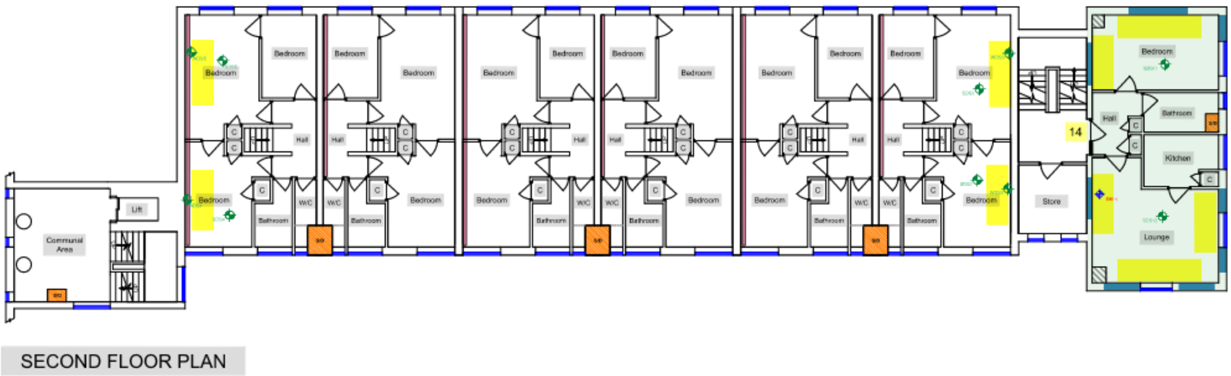


Figure 9 – Second Floor Plan of Charleton House *showing the flats inspected.*

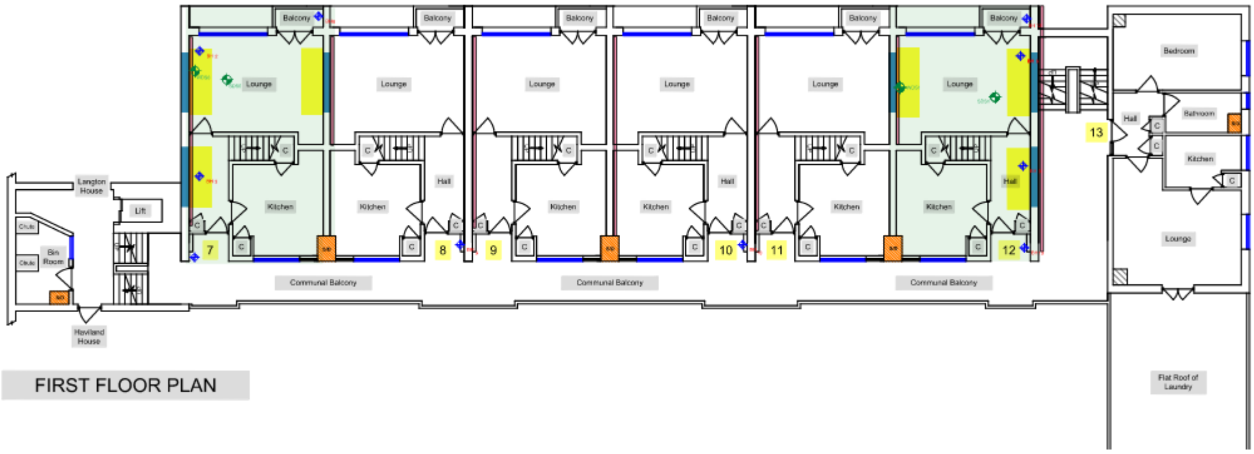


Figure 10 - First Floor Plan of Charleton House showing the flats inspected.

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 11 – 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 12 - 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 13 - Corroded balustrade connection.

- Loading bearing masonry wall construction confirmed as 300mm thick with a 22mm drill through the full depth and a 100mm core sample to half depth.

Figure 14 - Flat 14 brickwork external skin breaking out and 22mm depth check hole.

- Inconstantly installed tie reinforcement between wall panels and gothic beam floors slabs.

Figure 15 - Flat 20 flank wall inconsistently installed floor ties

- Infront of the flank walls and cross walls a 100mm thick shotcrete layer was found with 16mm ribbed reinforcement bars cast in. the reinforcement bars penetrated through the floor and was cast into the shotcrete above and below. The bars extended approximately 600mm above and below the floors but did not tie onto any other reinforcement within the shotcrete. The Shotcrete and reinforcement appear to be a remedial effort at providing a vertical tie within the building, however as there is no continuous reinforcement the vertical tie is not achieved.

Figure 16 - Shotcrete lining with reinforcing projecting through the floor.

- The underside of the ceiling adjacent to all walls has a tapering shotcrete layer with a minimum thickness of 25mm, thickening to 75mm at the wall face. The shotcrete encases 16mm reinforcement bars penetrating through the walls. The shotcrete and reinforcement is provided with the aim of providing the horizontal tie in the building, however as the reinforcement is not continuous across the floor the horizontal tie is not achieved.

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.

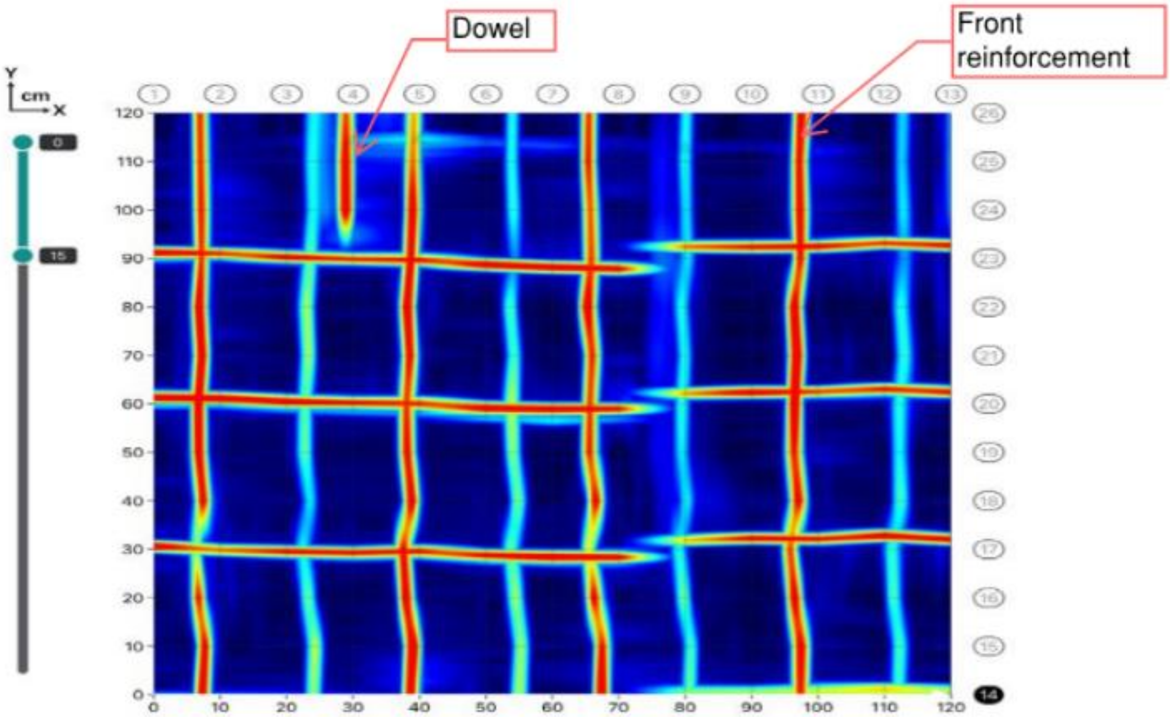


Figure 17 – Sample of GPR Scan results for a precast cross wall in Charleton House

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

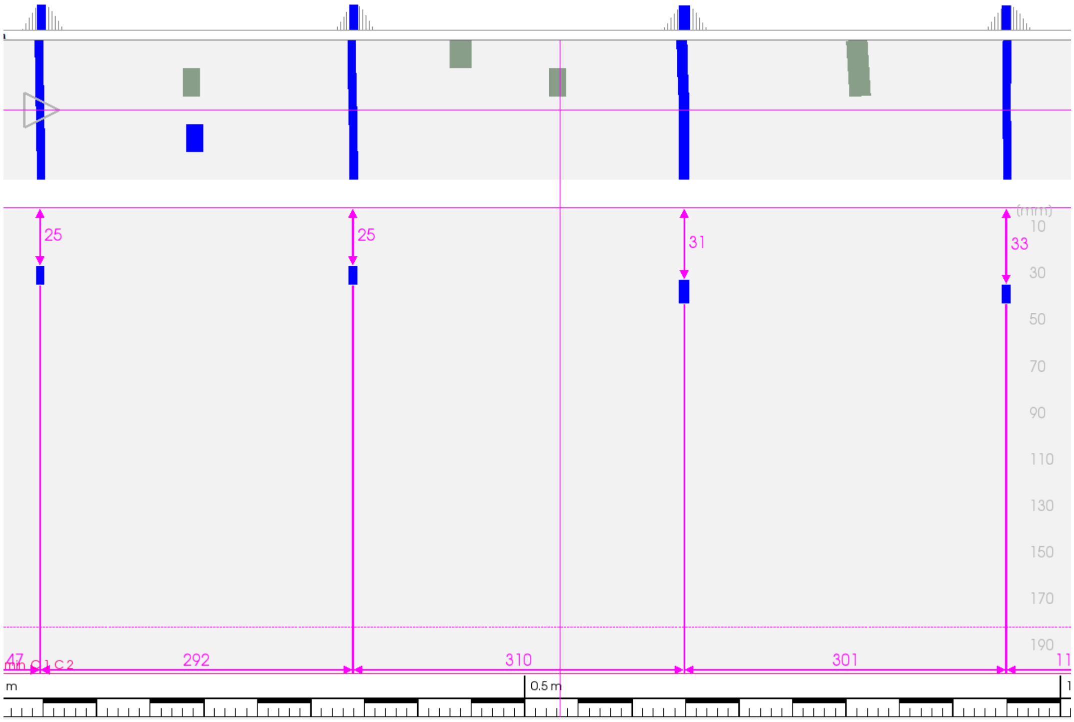


Figure 18 - Ferro scan of precast concrete cross wall showing reinforcement spacing and cover.

- 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 19 – Charleton House 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.

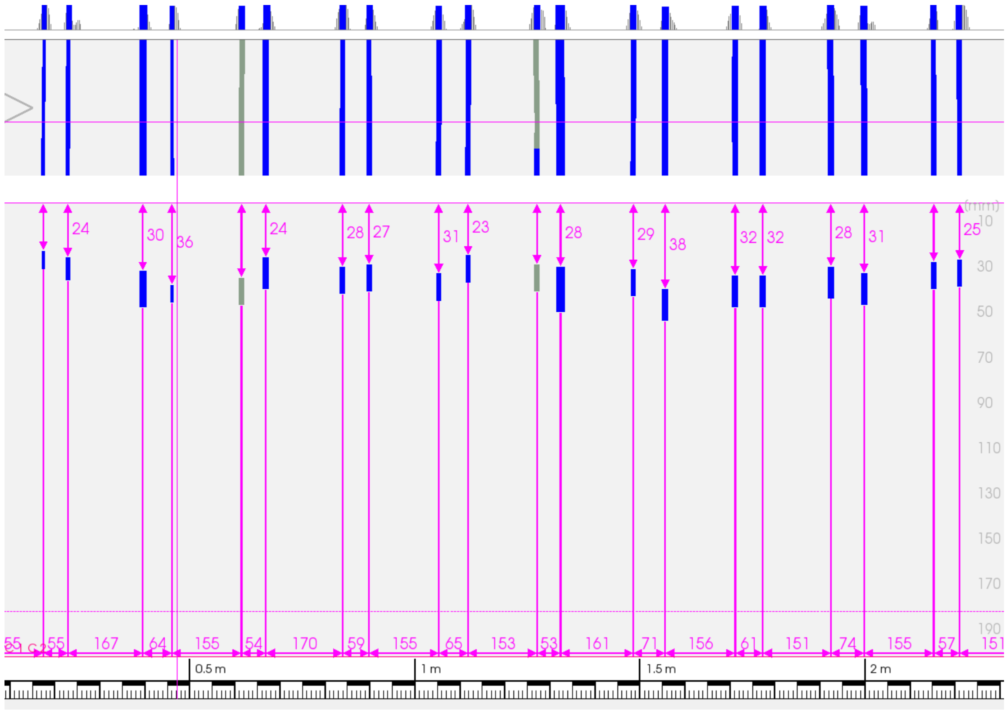


Figure 20 - Ferro scan of Charleton House floor soffit

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'.

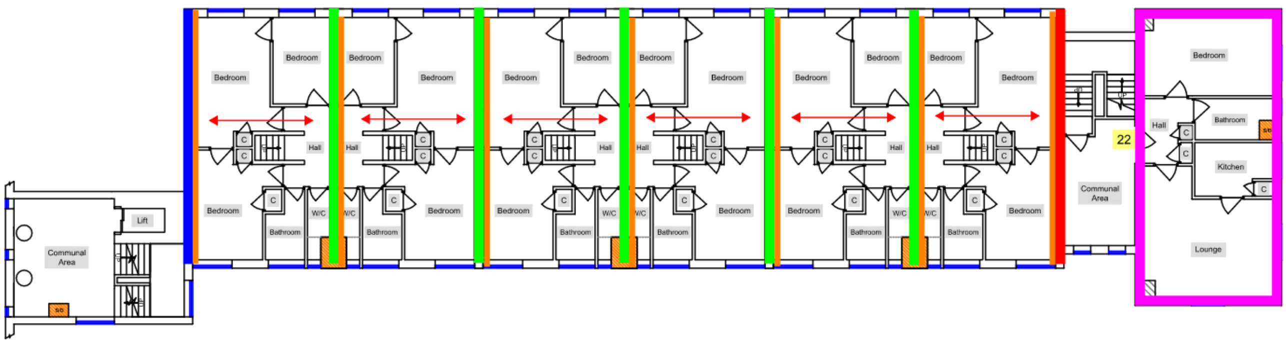


Figure 21 – Charleton House typical floor layout showing "key element" locations

The key elements, herein referred to as In-situ Flank Walls (shown in Blue), Precast Cross Walls (Green), Precast Flank Walls (Red), Load bearing masonry (Magenta), Shotcrete lining (Orange) and Floor Slabs (Red Arrows), 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: None – Mass concrete wall
Vertical Tie Reinforcement: 6mm ($^{1}/_{4}$") square twist bars @ 300mm c/c
Historic Strengthening: 100mm shotcrete wall with 12mm vertical tie bars through the floor at 600mm 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
Historic Strengthening: 100mm shotcrete wall with 12mm vertical tie bars through the floor at 600mm c/c

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
Additional Structure: 75mm lightweight blockwork wall to one side of wall.

Floor Slab Construction

Span: 5.042m (16′ 6.5″) max.
Construction: 190.5mm thick gothic beams with 50mm screed over.
Bottom Reinforcement: 10mm ($^3/_8$″) ribbed bars @ 150mm c/c
Tie Reinforcement: 10mm ($^3/_8$″) square twisted bars @ 200mm & 400mm c/c (900mm embedment into floor either side of wall. Ties are missing in many locations)

Load Bearing Masonry Wall Construction

Height: 2.465m
Construction: 300mm thick brickwork (Flemish Bond)
Vertical Tie Reinforcement: None

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:

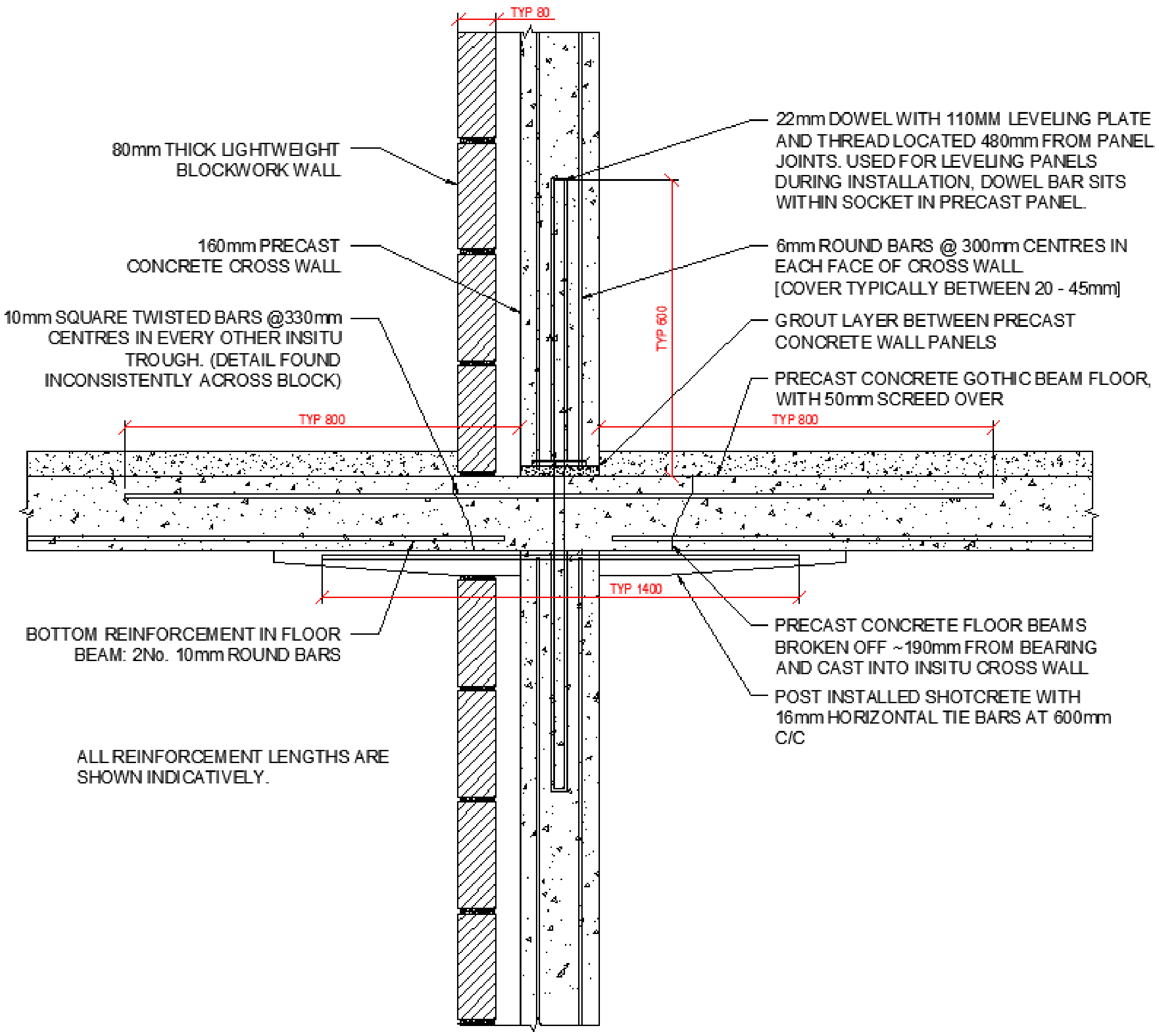


Figure 22 – Cross wall / floor slab joint

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 one 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. The two precast concrete panels are grouted together with a thin layer of grout. This can potentially act as a weak point in the wall panel system and was identified on site through the much lighter colour of the grouted layer.

Record drawings suggest that horizontal ties should have been provided in every trough between the gothic beams, the horizontal ties 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. The addition of the shotcrete and 16mm bars to the ceiling of at flat appears to have been done in order to provide a horizontal tie at the cross-wall junction. However, as the reinforcement is not continuous with primary reinforcement in the gothic beams, the horizontal tie force cannot be achieved.

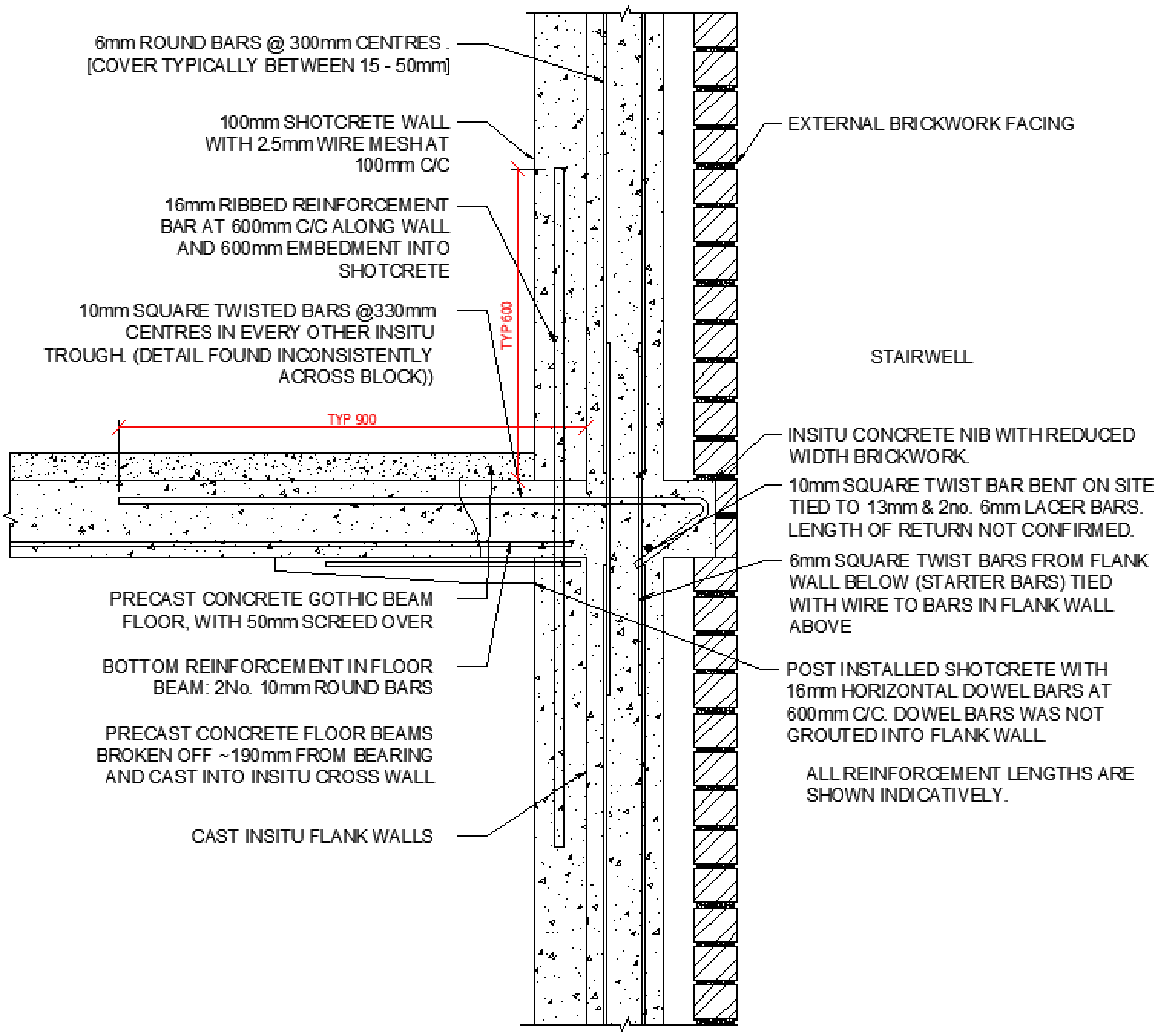


Figure 23 – Typical In-situ flank wall to floor slab joint

The investigations into the flank walls show that one flank wall is in-situ reinforced concrete and the other is precast as highlighted in Figure 20. The flank walls were found to both have a 100mm shotcrete liner with 2.5mm wire mesh reinforcement and 16mm dowel bars running vertically through each floor at 600mm c/c. These were assumed to be as a remedial measure with the attempt to provide a vertical tie in the building, however as the reinforcement does not lap correctly the detail is insufficient in providing the required tie.

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 bars.

The investigation into the precast flank wall / floor slab joint suggested the existence of horizontal tie provision with a similar detail of a 10mm square twisted bar being 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.

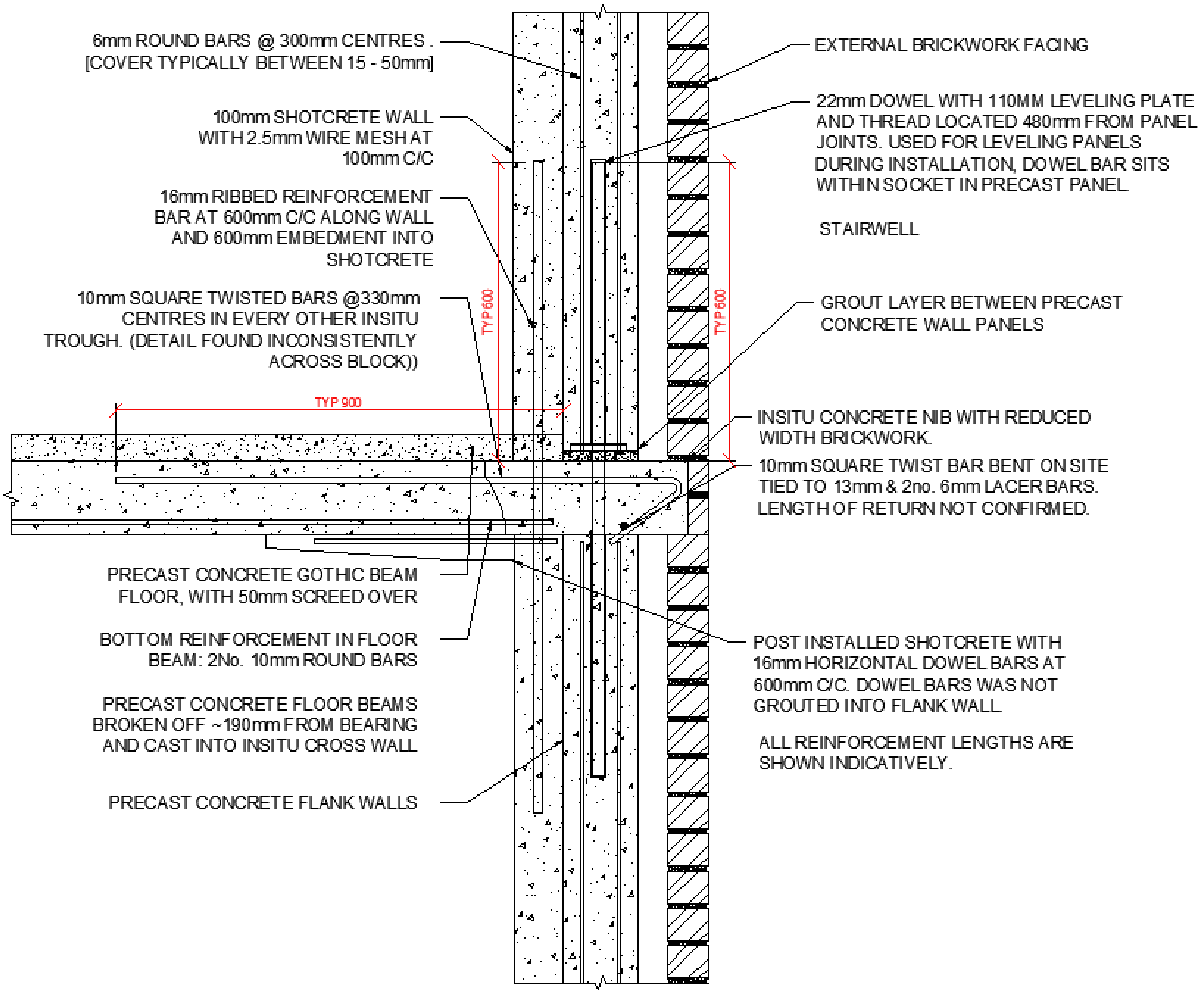


Figure 24 - Precast concrete flank wall/ floor slab joint

Wall Panel Vertical Joint

The following detail shown in figure 21 is a typical in-situ vertical joint between wall panels that would provide a horizontal tie between panel joints. 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. This detail was not indicated on the as built drawings for Charleton House, instead a simple unreinforced grouted butt joint was details, providing no horizontal tie between panels.

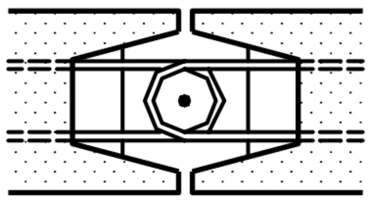


Figure 25 - Typical shear key connection at a vertical joint between wall panels

The existing Charleton House 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 25 below.

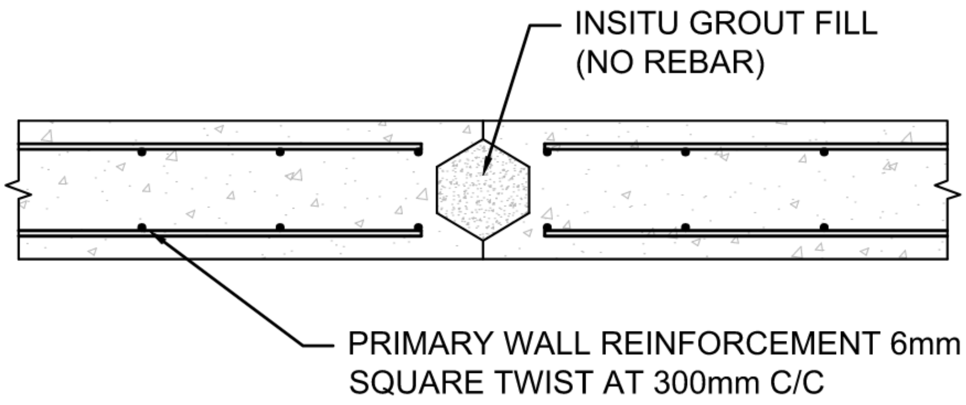


Figure 26 - Joint detail between cross wall panels as identified on site and in record drawings.

Floor Construction Detail

The floor slabs are constructed from 190mm thick 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 multiple troughs in a row. Furthermore, the ties appear to have been installed in pairs, with two troughs having reinforcement and then a gap of two toughs. 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 Charleton House, no tie bars were located.

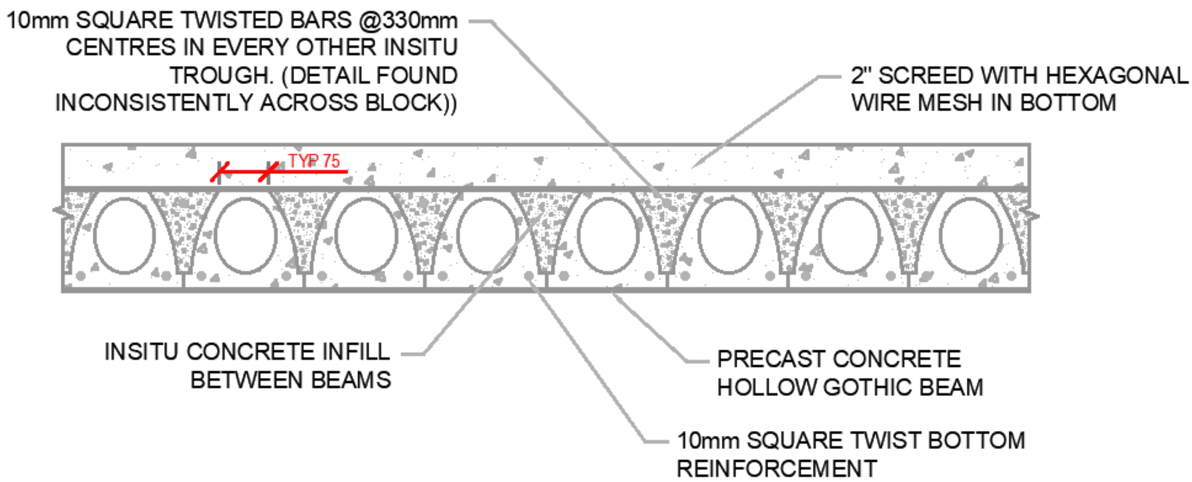


Figure 27 – Typical Gothic beam floor to wall joint construction detail.

4.5. Historic Strengthening

Following Ronan Point, Charleton House had additional strengthening works completed with the installation of shotcrete applied to walls and ceilings incorporating additional reinforcement. Figures 21, 22 & 23 show the additional shotcrete that has been applied to the flank walls and the shotcrete that has been applied to the ceilings at the wall junctions in each flat. Both instances utilise additional reinforcement passing either through the walls or floor slab and being encased in shotcrete to provide either a horizontal or vertical tie.

The vertical tie was installed by drilling 16mm reinforcement bars through the floor slabs at the face of the wall panels at 600mm c/c. These are then encased within the shotcrete above and below the floor. However, the shotcrete is unreinforced and therefore no continuity of reinforcement was found meaning an effective vertical tie is not achieved.

Horizontally a similar detail was installed with a downstand to the ceilings against the walls and 16mm bars passed through the walls encased on either side. Similarly to the vertical tie, because no reinforcement continuity was achieved the detail does not provide an effective horizontal tie.

4.6. Facade Wall

The external front and back walls that infill between the structural cross walls is 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.

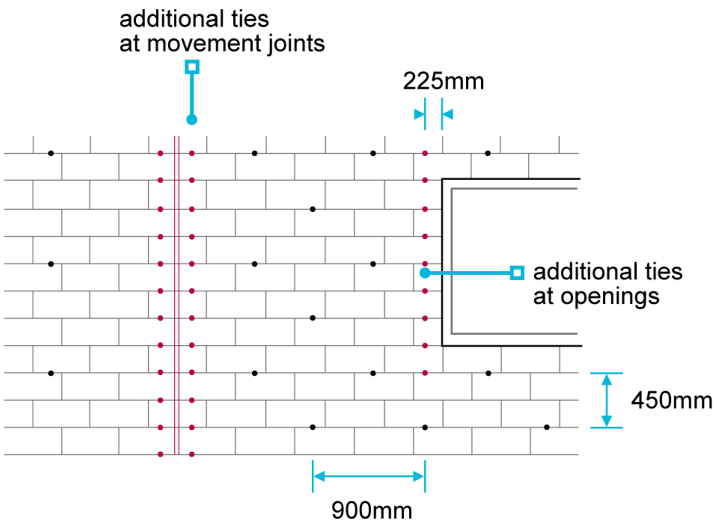


Figure 28 - Typical Wall tie detail (Ref NHBC Standard 6.1.18)

As indicated in Figure 29 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, in most cases these had not been used when building the walls. The intrusive investigations only allowed the use of the ties back to the precast panels in the inner skin, however considering the lack of use of the ties in the inner skin, plus the lack of cavity ties between the masonry skins the wall cannot be considered adequately tied.

Figure 29 - Removed blockwork showing missing or unused wall tie

5. STRUCTURAL FIRE ASSESSMENT

Charleton House is structurally linked to Havilland house via a stair core, should Charleton House loose load bearing capacity and fail within the 90-minute load bearing period of Haviland House, it is deemed to structurally impact the escape route of Haviland House. Therefore, the more onerous 90-minute fire period is applied in this assessment.

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

- Floor Soffit Cover – 12mm – 25mm from breaking out, and 17mm on average from ferro scans.
- Flank Wall Cover – 49mm - 50mm from ferro scans.
- Cross Wall Cover – 22mm - 25mm from breaking out, 25mm to 46mm from ferro scans; 22mm on average taken from breaking out as this is seen as both more reliable and more conservative.

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			
	μ_{fi} = 0,35		μ_{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 30 - 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.

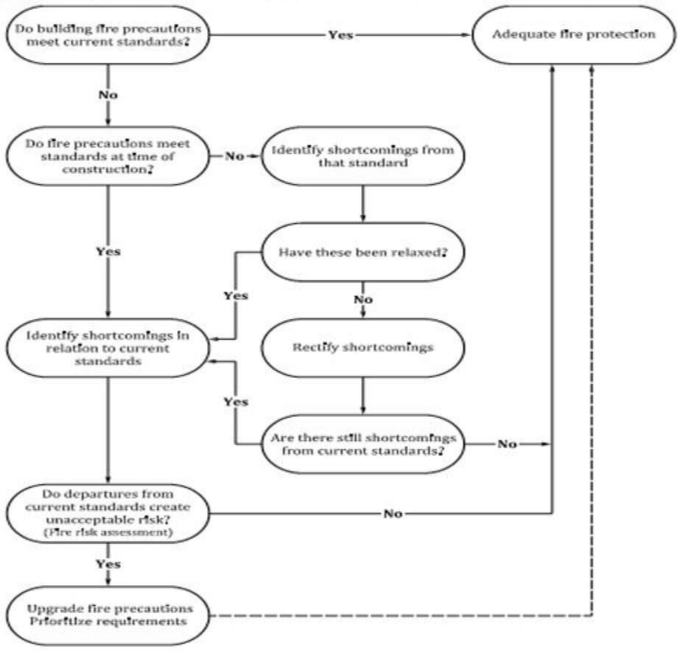


Figure 31 – Decision tree in relation to the requirement to upgrade fire precautions

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.

It is worth noting, the Eurocode 2 guidance, that informs these calculations, is for solid slabs of minimum 200mm thickness. By contrast, in Charleton House 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.

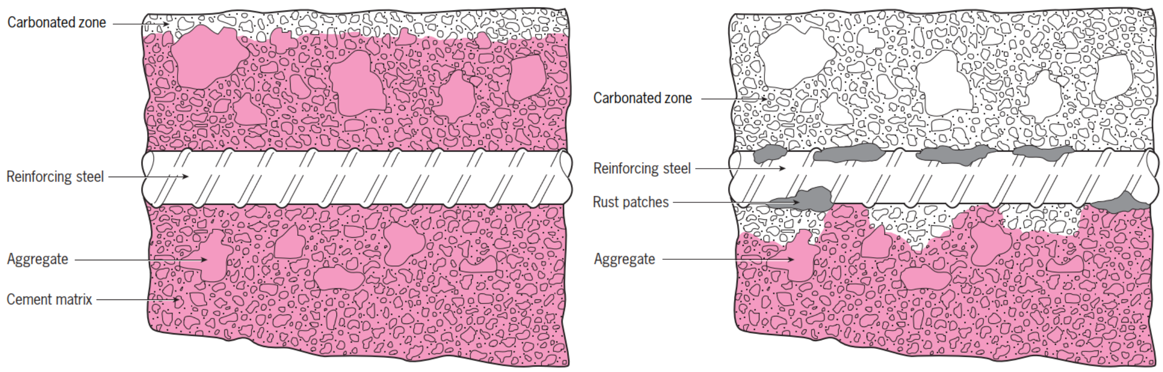


Figure 32 – (Left) Diagrammatic view of steel protected from carbonation-induced corrosion in partially carbonated concrete, (Right) Diagrammatic view of steel corroding in carbonated concrete.

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 3 - Carbonation Depths

TEST REFERENCE	TEST LOCATION	MEMBER TYPE	CARBONATION DEPTH	MIN. DESIGN COVER TO BAR	CARBONATION SURPASSED REINFORCEMENT?
SDS1	Flat 12	Slab	0-5mm	12.5mm	Yes*
SDS2	Flat 12	Slab	0-5mm	12.5mm	Yes*
SDS3	Flat 12	Slab	5-10mm	12.5mm	Yes
SDS4	Flat 7	Slab	5-10mm	12.5mm	Yes
SDS5	Flat 7	Slab	5-10mm	12.5mm	Yes
SDS6	Flat 7	Slab	0-5mm	12.5mm	Yes*
SDS7	Flat 20	Slab	5-10mm	12.5mm	Yes
SDS8	Flat 20	Slab	5-10mm	12.5mm	Yes
SDS9	Flat 20	Slab	5-10mm	12.5mm	Yes
SDS10	Flat 20	Slab	5-10mm	12.5mm	Yes
SDS11	Flat 14	Slab	0-5mm	12.5mm	Yes*
SDS12	Flat 14	Slab	5-10mm	12.5mm	Yes
WDS1	Flat 12	Precast Wall	5-10mm	25-30mm	No
WDS2	Flat 12	In Situ Wall	5-10mm	25-30mm	No
WDS3	Flat 12	In Situ Wall	5-10mm	25-30mm	No
WDS4	Flat 7	In Situ Wall	0-5mm	25-30mm	No
WDS5	Flat 7	In Situ Wall	5-10mm	25-30mm	No
WDS6	Flat 7	In Situ Wall	0-5mm	25-30mm	No
WDS7	Flat 20	In Situ Wall	5-10mm	25-30mm	No
WDS8	Flat 7	In Situ Wall	5-10mm	25-30mm	No
WDS9	Flat 7	In Situ Wall	5-10mm	25-30mm	No
WDS10	Flat 7	Precast Wall	0-5mm	25-30mm	No

Notes:

- * Carbonation of the concrete can be up to 10mm beyond that indicated by the test, therefore potential for carbonation of concrete around reinforcement.

- 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.

The carbonation depth within the wall elements was not observed to have surpassed the depth of the embedded reinforcement in any of the test locations completed in the flats of Charleton House. Meaning the rebar is within a passive environment and should therefore have sufficient protection from the concrete to prevent corrosion. Intrusive investigations competed in the stairwell on the outside of the two flank walls in Charleton House indicated that the carbonation depth had surpassed the depth of embedded reinforcement. This was due to a lower cover on the tie reinforcement in the flank walls within the brickwork support corbel. The reinforcement within the corbel is therefore not within a passive environment and does not have adequate protection to prevent corrosion. The carbonation depths observed did not appear to be excessive for a structure of this age. 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 and the effect of carbonation will vary from location to location depending on the cover.

The carbonation depth within the floor elements 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.

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.

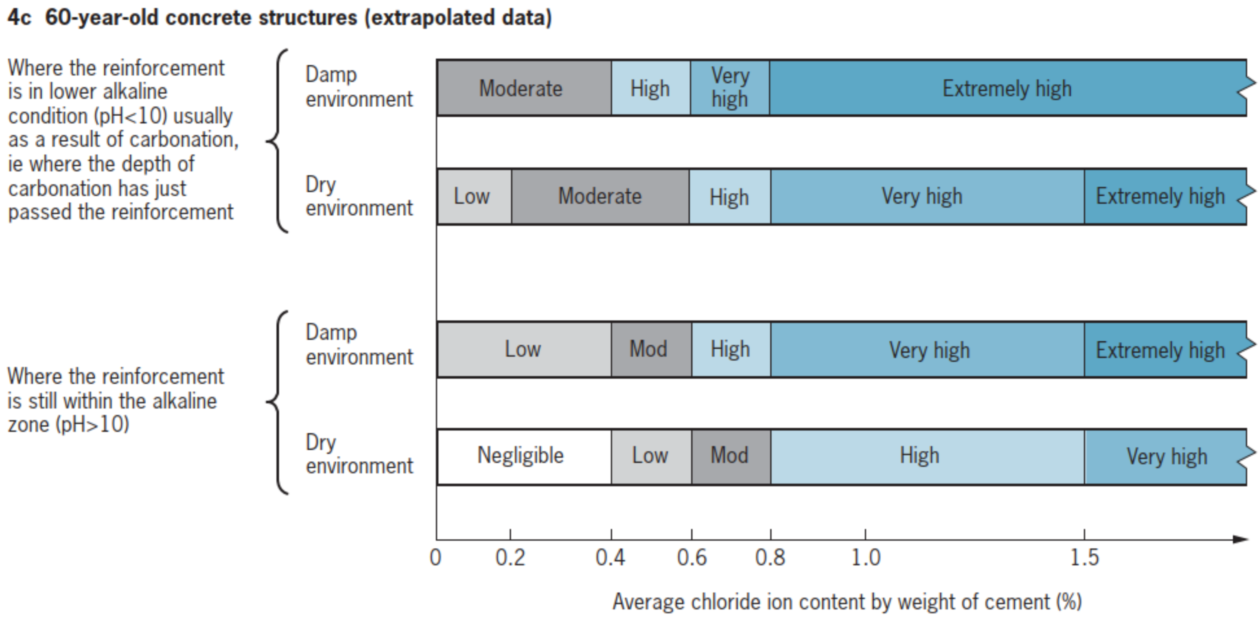


Figure 33 – Estimated risk of corrosion associated with carbonation, chloride content and environment.

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)
SDS1	Flat 12	Slab	Yes*	0.18	Low
SDS2	Flat 12	Slab	Yes*	0.18	Low
SDS3	Flat 12	Slab	Yes	0.16	Low
SDS4	Flat 7	Slab	Yes	0.18	Low
SDS5	Flat 7	Slab	Yes	0.16	Low
SDS6	Flat 7	Slab	Yes*	0.16	Low
SDS7	Flat 20	Slab	Yes	0.14	Low
SDS8	Flat 20	Slab	Yes	0.18	Low
SDS9	Flat 20	Slab	Yes	0.16	Low
SDS10	Flat 20	Slab	Yes	0.14	Low
SDS11	Flat 14	Slab	Yes*	0.16	Low
SDS12	Flat 14	Slab	Yes	0.18	Low
WDS1	Flat 12	Precast Wall	No	0.24	Negligible
WDS2	Flat 12	In Situ Wall	No	0.06	Negligible
WDS3	Flat 12	In Situ Wall	No	0.20	Negligible
WDS4	Flat 7	In Situ Wall	No	0.02	Negligible
WDS5	Flat 7	In Situ Wall	No	0.16	Negligible
WDS6	Flat 7	In Situ Wall	No	0.14	Negligible
WDS7	Flat 20	In Situ Wall	No	0.14	Negligible
WDS8	Flat 7	In Situ Wall	No	0.18	Negligible
WDS9	Flat 7	In Situ Wall	No	0.16	Negligible
WDS10	Flat 7	Precast Wall	No	0.18	Negligible

- * Carbonation of the concrete can be up to 10mm beyond that indicated by the test, therefore potential for carbonation of concrete around reinforcement.

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

- The chloride contents in the slab members was found to be low, with a negligible level found in the wall elements.

- Concrete members tested were found to have a negligible or low risk of corrosion to the embedded reinforcement. As stated in the previous subsection, it appears the elements which are at the 'low' risk of corrosion are so mainly due to the low concrete cover to the rebar and not a higher level of carbonation or the concrete.

- During the opening up works some degree of corrosion (typically minor surface corrosion) was noted to the reinforcement. This is likely to have been due to inadequate storage of the bars prior to installation (historic corrosion), because of the low risk of reinforcement corrosion.

- Due to the low & negligible risk levels of reinforcement corrosion the risk of concrete spalling is also considered to be low.

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 · SiO_2 and CaO · 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	22.4	40.8
Wall (In-situ)	3.4	18.6	43.8
Slab	3.3	25.2	39.1

Table 6 - Interpretation of cement composition testing

TEST MEMBER	SIO2	CAO	TOTAL (SIO2 + CAO)
Wall (Precast)	6.1%	54.9%	61%
Wall (In-situ)	7.8%	42.5%	50.3%
Slab	8.4%	64.2%	72.6%

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 (Precast)	61.0 < 66.6 ∴ FAIL	9.0 > 2.0 ∴ PASS
Wall (In-situ)	50.3 < 66.6 ∴ FAIL	5.5 > 2.0 ∴ PASS
Slab	72.6 > 66.6 ∴ PASS	7.6 > 2.0 ∴ PASS

By inspection of the interpreted results, the quantities of Silica (SiO_2) and Calcium Oxide (CaO) do not satisfy the expected proportions for today's standards. The concrete is therefore likely to not be offering adequate protection for embedded reinforcement. The cement consisting of less than 2/3rds Silica (SiO_2) and Calcium Oxide (CaO) may result in a number of possible issues including, lower strength, increase possibility of carbonation, poor workability and decreased resistances to high temperatures (low CaO contents). Some of these factors do not directly impact the building as installed and through further testing we have been able to confirm other characteristics including the strength of the concrete.

The reduced resistance to high temperatures due to the Low CaO content will need to be considered as part of the fire resistance checks on the building. However, most of the wall panels have either a shotcrete lining with plasterboard finish or a blockwork liner with plasterboard finish. This will provide an additional layer of protection to the wall panels in the event of a fire in the flats.

6.4. Compressive Strength

In order to assess the robustness of the concrete elements forming Charleton 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 Charleton House 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 Charleton House

REF	LOCATION	CORE LOCATION	COMPRESSIVE STRENGTH
BH1	Flat 14	Brick Wal	31.1 N/mm^2
BH2	Flat 7	Flank Wall	50.2 N/mm^2
BH3	Flat 7	Flank Wall	34.2 N/mm^2
BH4	Flat 7	Cross Wall	38.3 N/mm^2
BH5	Flat 8/9	Cross Wall	48.9 N/mm^2
BH6	Flat 10/11	Cross Wall	48.0 N/mm^2
BH8	Flat 12	Flank Wall	38.6 N/mm^2
BH9	Flat 12	Flank Wall	41.9 N/mm^2
BH10	Flat 12	Flank Wall	60.2 N/mm^2
BH11	Flat 20	Flank Wall	55.7 N/mm^2
BH12	Flat 12	Flank Wall	53. N/mm^2
BH13	Flat 16/17	Cross Wall	63.7 N/mm^2
BH14	Flat 18/19	Cross Wall	64.2 N/mm^2
BH16	Flat 20	Flank Wall	52.7 N/mm^2
BH17	Flat 20	Flank Wall	34.8 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 in Charleton House is:

Table 9 - Compressive strength results for Charleton House Cozens

TEST LOCATION	COMPRESSIVE STRENGTH
Cross Wall	**27.7N/mm^2**
Flank Wall	**24.69N/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.

7. CHARLETON STRUCTURAL ASSESSMENT

7.1. Assessment Criteria

The Charleton House 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 Flank wall / floor slab joint has shown that:
<u>Horizontal Ties:</u> Insufficient due to inconsistencies
<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. Furthermore, due to the lack of grout between the retrofit dowel bar and flank wall, no additional horizontal tie is achieved. Any tie force here would also rely on the friction achieved between the shotcrete and underside of the gothic beams as there is no lap of the reinforcement.

No reinforcement runs continuously from the top of one panel into the bottom of the next, the dowels are connected via the 22mm dowel bars that are not fully grouted. Therefore, no continuous vertical tie is provided, **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 Flank wall / floor slab joint has shown that:
<u>Horizontal Ties:</u> Insufficient due to inconsistencies
<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 **insufficient** to pass the assessment for Consequence Class 2b.

Flank Wall / Floor Slab Joints (Masonry)

The assessment of the Flank wall / floor slab joint has shown that:
Horizontal Ties: Insufficient
Vertical Ties: Insufficient

No ties are present between the precast gothic beam floors and the masonry walls. **The existing provision of horizontal ties does meet the requirements of the resisting the minimum tie force.**

Continuous vertical reinforcement was not found within the masonry build up and from an assessment of the walls panels they cannot resist a 17kN/m^2 loading to be designed as key elements.

The masonry flank wall joint is therefore **insufficient** to pass the assessment for Consequence Class 2b.

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 34 below, to prevent the joint from uncontrolled separation.

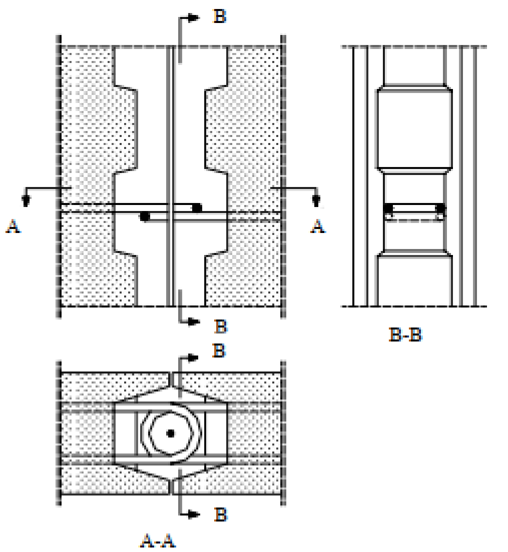


Figure 34 - Typical shear key connection at a vertical joint between wall panels

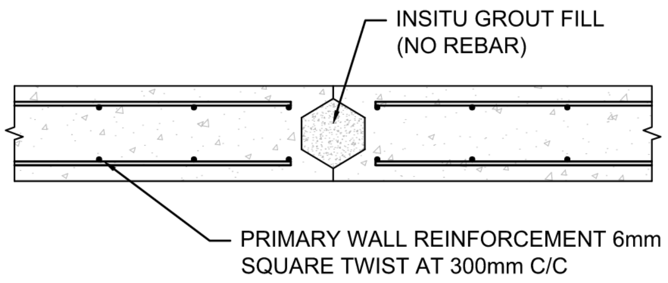


Figure 35 - Joint detail between cross wall panels as identified on site & in as built drawings.

However, within Charleton House, 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 – Charleton House tie details summary table

Charleton House (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
Masonry Wall	X	X	Insufficient
Wall to Wall Joints	X	X	Insufficient*

Charleton House – 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, although it should be noted that latest British Standards (now withdrawn) are more akin to the design codes that the structure would have been originally designed to.

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 **concrete** structural elements that form Charleton House, **with the exception of flank and cross walls at Level 1, are insufficient** to resist a loading of this magnitude. The **masonry** (external brick) **walls** that form Charleton House annex, with the **exception of Level 5, are sufficient** to resist a loading of this magnitude.

The following table summarises the findings:

Table 12 – Charleton House robustness assessment summary table

STRUCTURAL ELEMENT	CHARLETON HOUSE 17KN/M2 OVERPRESSURE (NO PIPED-GAS SUPPLY)	NOTES
Floor Slab (Downward)	X	Inadequately Robust
Floor Slab (Uplift)	X	Inadequately Robust
Flank Wall Level [2-5]	X	Inadequately Robust
Flank Wall Level 1	a	Adequately Robust
Cross Wall Level [2-5]	X	Inadequately Robust
Cross Wall Level 1	a	Adequately Robust
External Brick Wall Level 5	X	Inadequately Robust
External Brick Wall Level [1-4]	a	Adequately Robust

Concrete wall panels are adequately robust below the 2rd floor where the axial load imposed by the weight of the building provides a favourable scenario against failure. At levels 2-5 the wall panels fail under the 17kN/m² overpressure loading.

Masonry wall panels are adequately robust from floors 1-4 due to the axial load imposed by the weight of the walls above increasing the flexural strength of the panel and provides a favourable scenario against failure. Above the 4th floor the wall panels will fail under the 17kN/m² overpressure loading, however this will only lead to a failure of the roof and will therefore not result in a disproportionate collapse scenario.

Charleton House – LPS Criterion 2 – Adequate Collapse Resistance
Masonry Panels Satisfactory - Adequate Collapse Resistance
Concrete Frame 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.

Charleton House – 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.

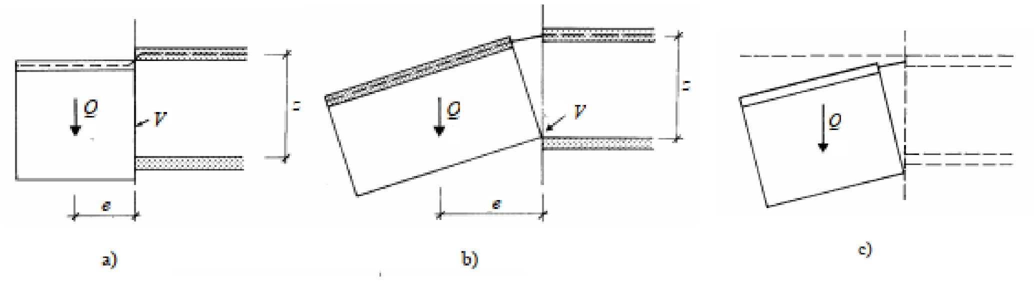


Figure 36 - Examples of collapse mechanisms for a cantilevering wall above a damaged area, a) joint slip mechanism, b) rotation mechanism, c) combined slip-rotation (fib, 2008)

The load bearing masonry portion of Charleton House will likely have the ability to mobilise alternative load paths through the ability of the masonry to arch over a failure point. The failure mechanism of a masonry wall panel in an over pressure event would be the localised failure of a small portion of the wall, the remaining masonry panel will likely be able to redistribute loads and arch over the failed section of the panel.

7.5. Summary of LPS Criteria Checks

Charleton House 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. This, however, does not apply in the same way to the loadbearing masonry annex, which performs better in Criterion 2 compared to the concrete block. Furthermore, the masonry is expected to only fail locally, which would leave a large part of the panel in place allowing vertical load transfer via arching over an opening. The masonry panel is also able to resist the over pressure up to the 4th floor meaning a disproportionate collapse scenario would not occur as only the roof would fail.

	Charleton House – Main Block	
LPS CRITERION	**PASS / FAIL**	**NOTES**
LPS 1	X	Inadequately Tied
LPS 2	X	Inadequately Robust
LPS 3	X	Inadequate Mobilisation of Alternative Load Paths
	Charleton House – Conclusion	
	Insufficient	

	Charleton House – Masonry Annex	
LPS CRITERION	**PASS / FAIL**	**NOTES**
LPS 1	X	Inadequately Tied
LPS 2	a	Adequately Robust
LPS 3	a	Adequate Mobilisation of Alternative Load Paths
	Charleton House – Conclusion	
	Sufficient	

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 shared access walkways on the front 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. As stated in previous subsections, it appears the elements which are at the 'low' 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 24.69N/mm², and 27.7N/mm², 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.

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 link around a 13mm lacer bar in the flank wall and extend 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.
- A grout layer was identified between the precast wall panels used to bed the panels together when installing.
- No lateral tie has been found at precast panel vertical joints with joints being grouted only.
- A 100mm shotcrete remedial detail has been applied to the flank walls across the building incorporating 16mm vertical bars passing through the floor in an attempt to provide a vertical tie. Internal walls have a blockwork lining consisting of 75mm lightweight blockwork with no ties back to the precast panel.
- 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 Charleton House 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 given the shared lift and stair core with Havilland House. 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 balustrades.

However, under accidental loading the concrete section of the structure is assessed to be inadequate in the event of a non-piped gas explosion, whilst the masonry annex is adequate in this scenario. 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 Charleton House, 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. 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.

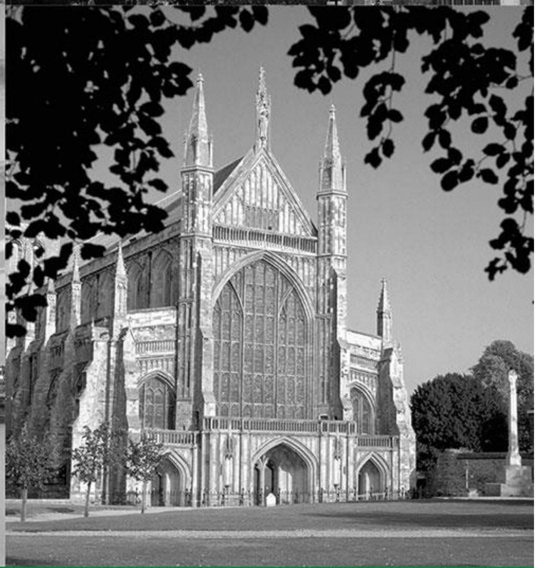
2. Visual inspections at 10-year intervals to structural joints which are vulnerable to water penetration; locations such as flank walls and roofs.

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

Should the risk reduction measures proposed not effectively limit the residual 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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