RIDGE

ST JUDES – LANGTON HOUSE
STRUCTURAL ROBUSTNESS ASSESSMENT

BRISTOL CITY COUNCIL
January 2025

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STRUCTURAL ROBUSTNESS ASSESSMENT

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Prepared for

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VERSION CONTROL

PROJECT NAME: St Jude's – Langton House Structural Robustness Assessment
PROJECT NUMBER: 5025719
DOCUMENT REFERENCE: 5025719-RDG-XX-XX-T-S-009904

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P02	19.02.2025	Draft Final Issue	Robert Hurley Meng (Hons)	William Morris BEng (Hons) CEng MIStructE	James McCulloch BEng (Hons) MSc CEng MICE
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1. EXECUTIVE SUMMARY

Intrusive investigations were conducted on the dwelling blocks at St Jude's, 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 Langton House, unlike other buildings at St Jude's is an in-situ reinforced concrete frame building with reinforced walls and floors. The findings from intrusive investigations suggested that the building should not be susceptible to disproportionate, progressive collapse and does 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 residential 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 the Approved Document A requirements for structural design and detailing for a consequence class 2b building requiring the provision of horizontal and vertical ties as defined in section 5.2 & 5.3 of the approved document. As Langton House shares its single route of escape through the lift and stair core structure with Haviland House, the potential to impact this structure, the fire assessment criteria and requirements for this have been applied to Langton House.

Table 1 - Summary of Approved Document A Criteria for Langton House

LPS CRITERION	ASSESSMENT	COMMENTS
Approved Document A - Criterion 1 Adequate Vertical ties within joints	**Sufficient**	Langton House is a class 2B building that requires vertical ties. The vertical ties were found to be sufficient to withstand the required imposed forces.
Approved Document A - Criterion 2 Adequate Horizontal ties within joints	**Sufficient**	Langton House is a class 2B building that requires horizontal ties. The horizontal ties were found to be sufficient to withstand the required imposed forces.

Table 2 - Assessment criteria summary for Langton 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 support and tie details	**Insufficient**	The external masonry walls on the building, consisting of two layers of blockwork, was found to be inadequately tied to the primary concrete walls and floors and have insufficient 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 with instances of temporary propping being used to support the balustrades. 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 Langton House were noted to have spalling concrete on the underside.

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. Some measures for Langton House are incorporated from the taller Haviland House as it shares the lift core and stair structure with Haviland House:

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. Full condition survey of the balustrades around Langton House, temporary support provided to those in a critical condition with a design and programme developed to replace all the balustrades.
5. 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.
6. 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. Repairs and or replacement of the residential balconies due to deterioration from carbonation.
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 includes Langton House, Charlton House, Haviland 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 Langton 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 Langton House.

This report documents the main findings of the investigation and the findings of the subsequent structural assessment into the robustness of the Langton 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. Three suitable flats were identified, namely flats 11, 17 & 25.

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.

Langton House comprises of two-bedroom flats with and open plan living room and kitchen to the rear of each flat. The floors are constructed from a one-way spanning cast-insitu concrete slab with a concrete screed over the top. Being single storey flats no core samples of the flats have been obtained; however, rebound hammer tests have been used to understand the compressive strength of the concrete.

2.5. Purpose of Report

The purpose of the report is to advise on the insitu concrete construction of Langton 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

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

The dwelling block, Langton 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 as an insitu reinforced 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.

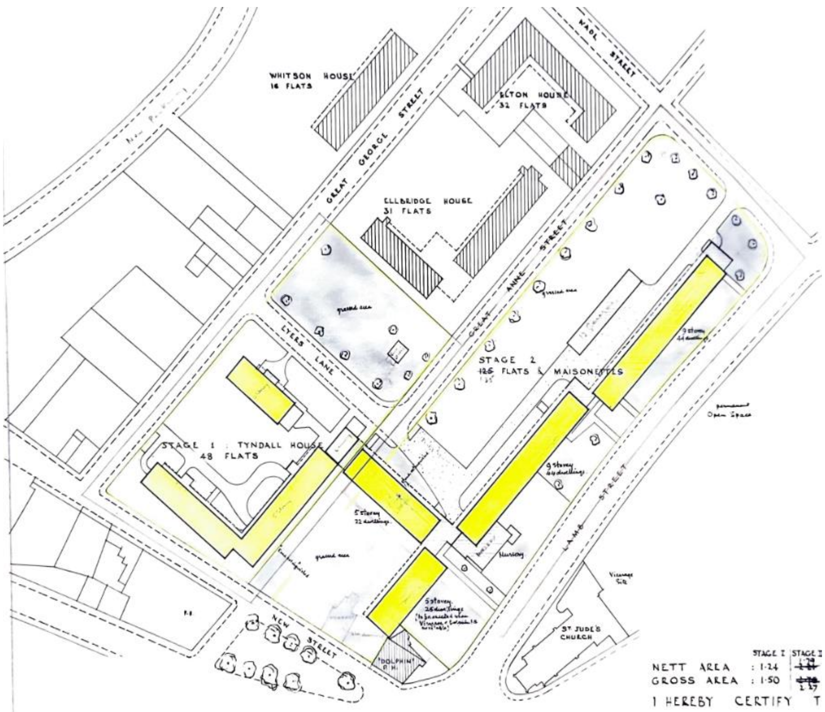


Figure 3 - As built site plan of Blocks A, B, C & D

The flats in Langton House are single storey flats containing two-bedroom with and open plan living room and kitchen to the rear of each flat.

Access to each of the flats is via a shared walkway along the front of the building formed by a cantilever section of the in-situ concrete walls, which supports a continuation of the one way spanning concrete floor slab. The access walkways are accessed via a lift & stair core at the one end of Langton House where it connects with the adjacent blocks.

Each flat has a balcony to the rear accessed through the living room, extending approximately 1000mm off the rear elevation of the building. The balconies on Langton House are a cantilevered continuation of the in-situ reinforced concrete floor slab.

4. INSPECTION & SURVEYS

4.1. Methodology of Assessment

The method used to carry out the assessment of the Langton House follows the hierarchical approach adopted by BRE 511 as shown in Figure 6 below. Whilst Langton House is not an LPS building the principles are relevant to the assessment of the in-situ concrete frame.

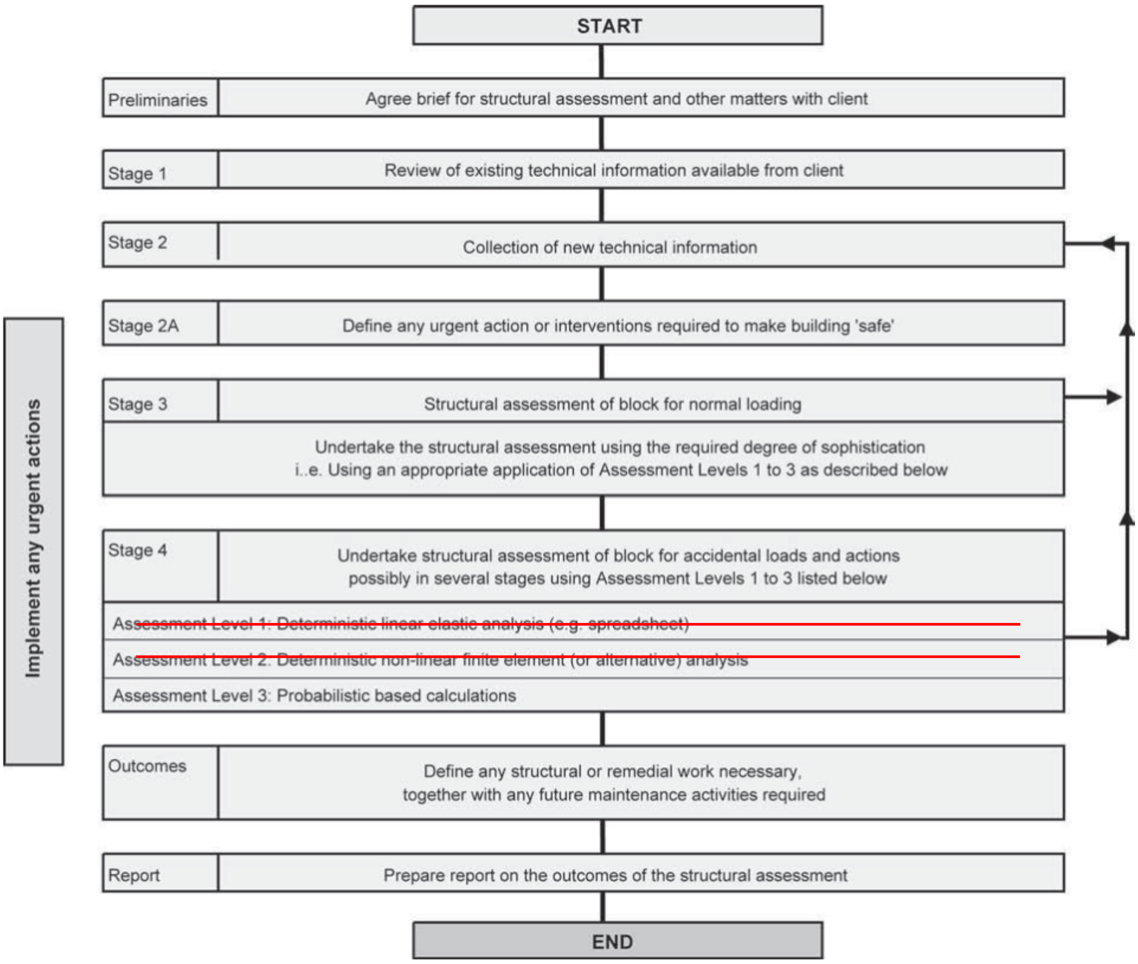


Figure 4 - Extract of BRE 511 figure 34 'Main steps in the structural assessment process'.

Three 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 11, 17 & 25 were subjected intrusive breaking out to confirm structural details, this gave a sample of 12% of the total number of flat in Langton House.

Using the limited As-Built information obtained from the construction details of the blocks Ridge subjected the three 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 5, 6 & 7 highlight the locations of the investigations undertaken within the block. Flats 11, 17 & 25 were subjected to the intrusive opening up works.

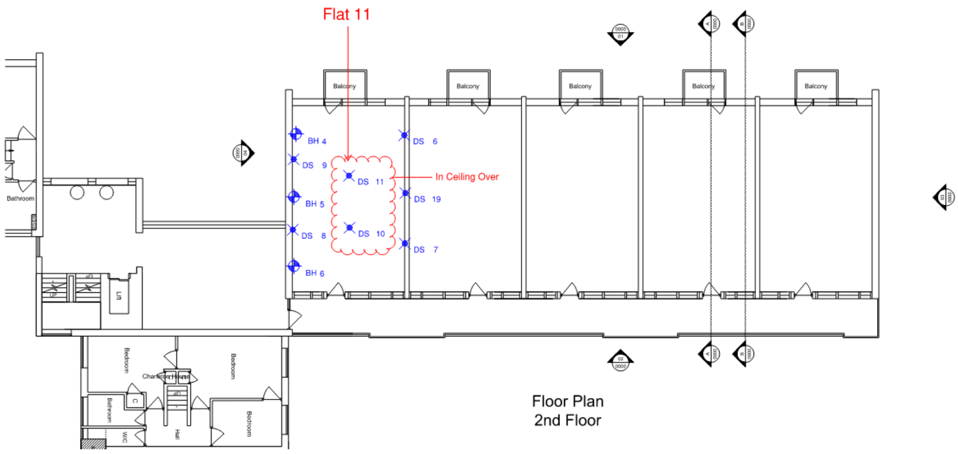


Figure 5 – Third floor plan for Langton House showing the flats inspected.

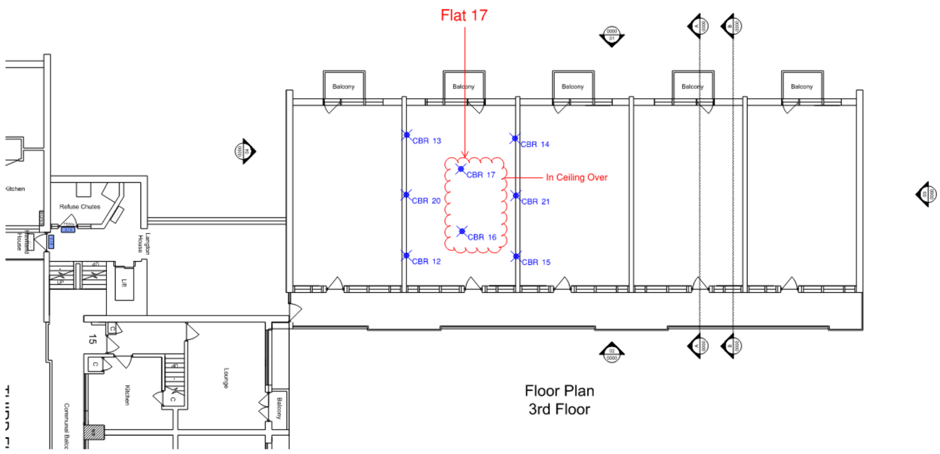


Figure 6 – Second Floor Plan of Langton House showing the flats inspected.

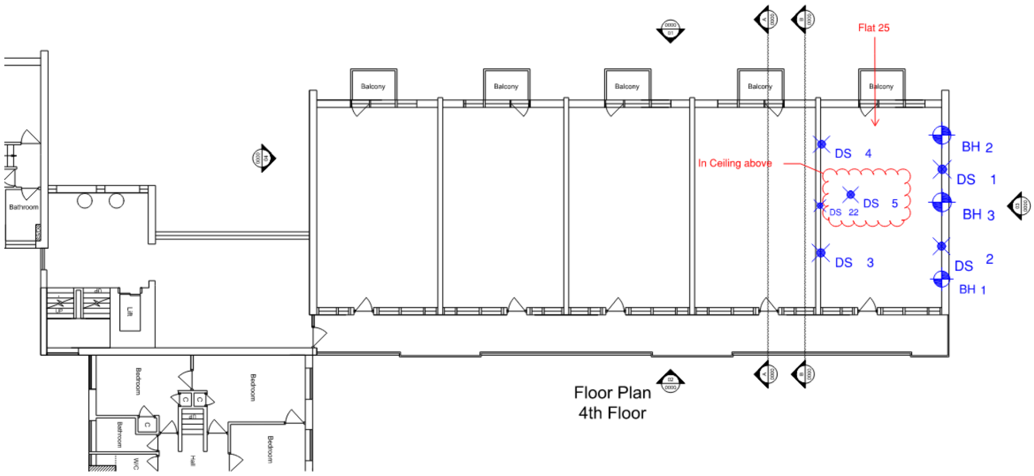


Figure 7 - First Floor Plan of Langton House showing the flats inspected.

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

The block is a reinforced in-situ concrete panel construction with one way spanning concrete slabs supported on in-situ concrete cross walls. The wall panels had a 6mm reinforcement in both directions and the slab had 6mm bars in the bottom at 90mm c/c and 10mm bars in the top and across joints.

Figure 8 - Floor to wall opening up works in Langton House

Figure 9 - Exposure of soffit reinforcement in Langton House

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.

Concrete cross walls:

- Cross walls were shown through GPR scans to have 2 layers of reinforcement at 300mm c/c.

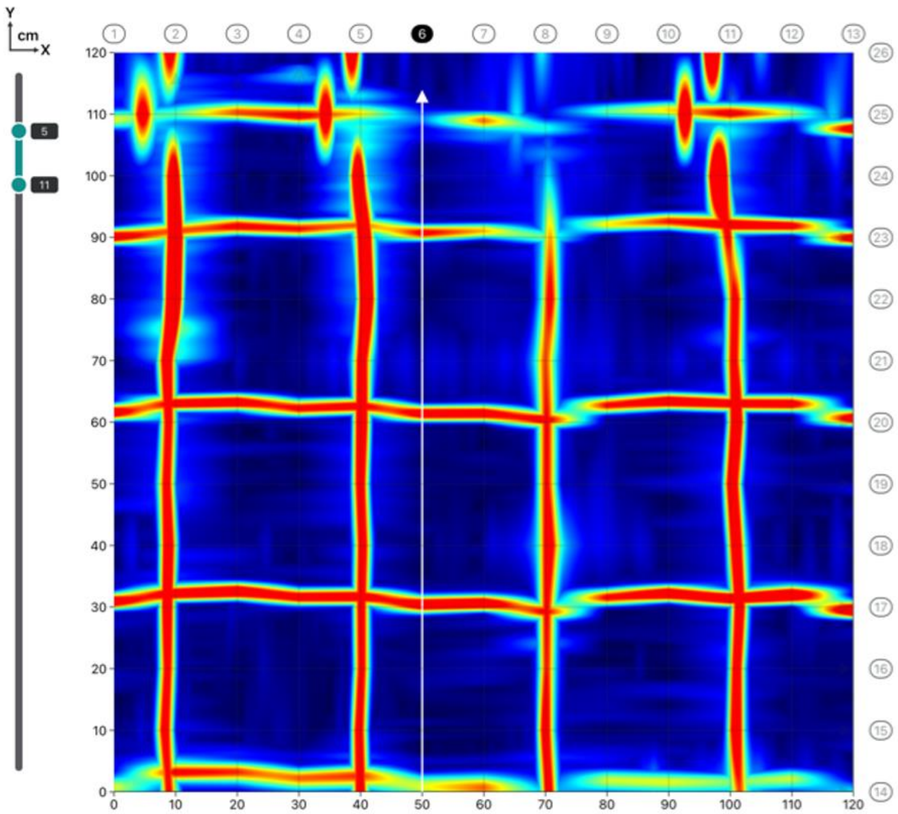


Figure 10 – Sample of GPR elevation scan results for a cross wall in Langton House

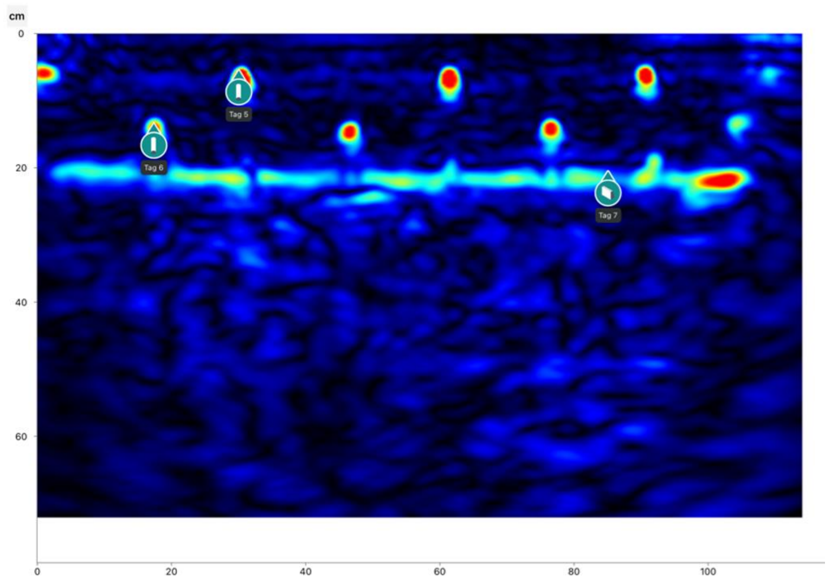


Figure 11 - Sample of GPR cross section scan results for a cross wall in Langton House

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

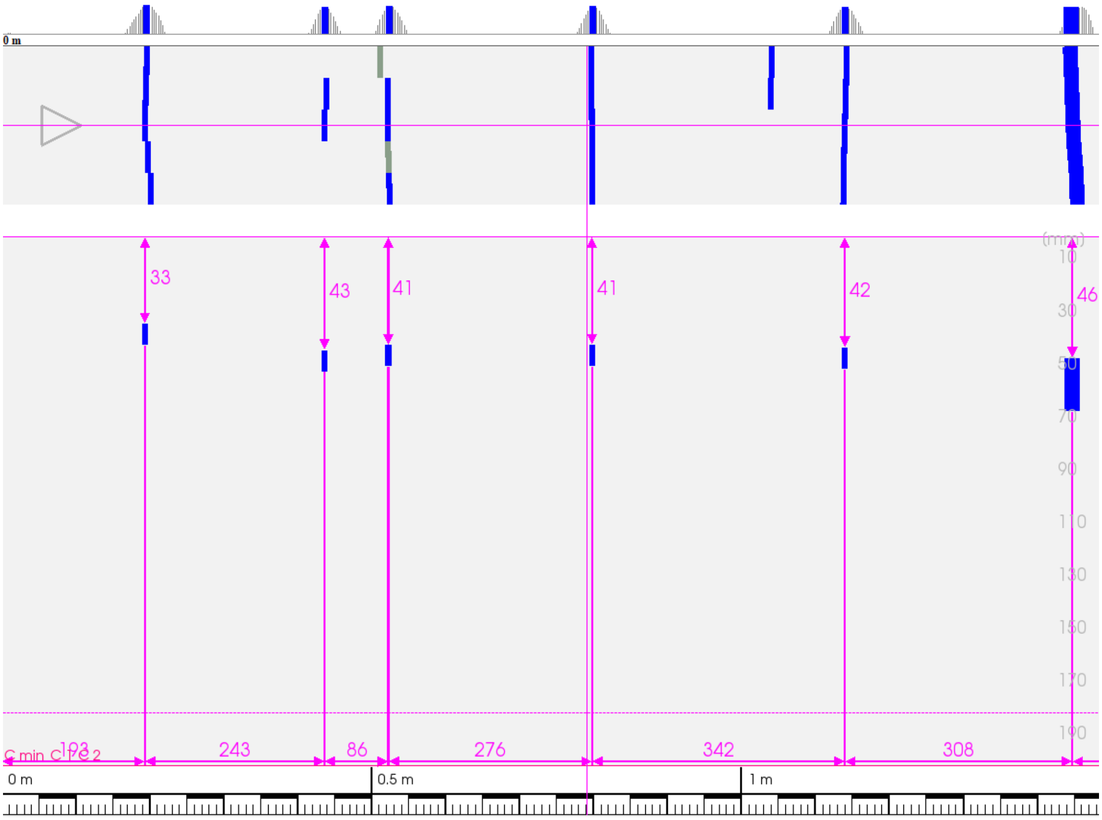


Figure 12 - Ferro scan of concrete cross wall showing reinforcement spacing and cover.

- Whilst the scanning of the reinforcement showed the spacing of the reinforcement to vary across the block from 250mm to 350mm, intrusive investigations showed the spacing to be between 300mm and 350mm with a bar diameter of 6mm. For analysis purposes an average of 300mm spacing has been used for the calculations.

Figure 13 – Langton House intrusive wall investigations

Concrete Slab:

- GPR scans of the top and bottom of the slab shows 8mm reinforcement bars at 90mm c/c in the bottom and 12mm bars at 550mm c/c in the top. It was noted from verification of the scans that they often indicate larger bars than those installed. Therefore, from the scans it is assumed the bars in the bottom are 6mm and the bars in the top are 10mm.

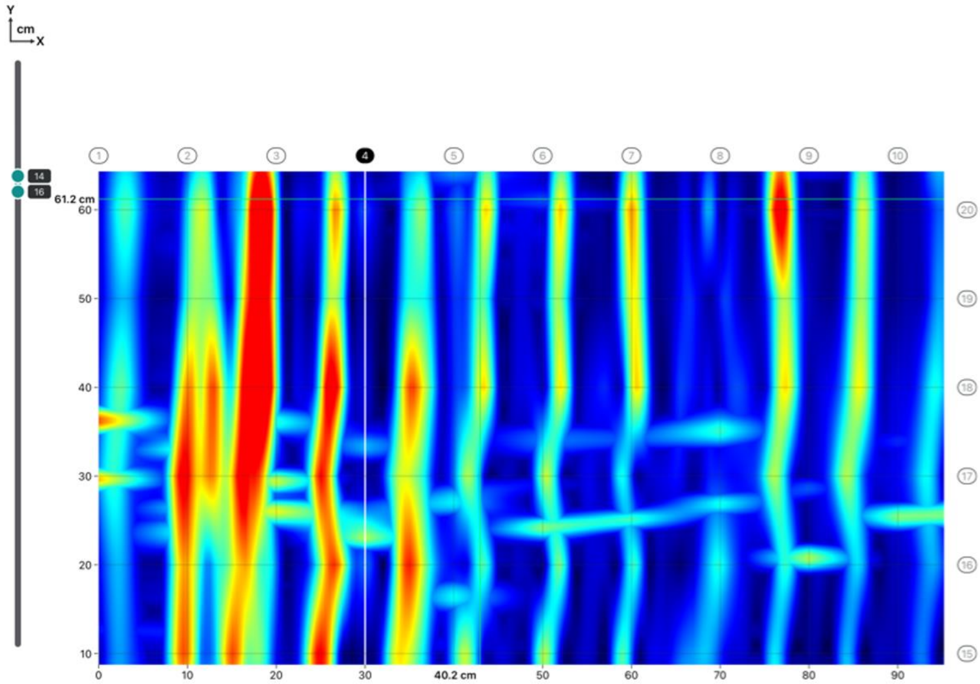


Figure 14 - GPR scan of Langton House Soffit

- The GPR scans correlate to the ferro scans completed and also give representation of the cover to the primary reinforcement.

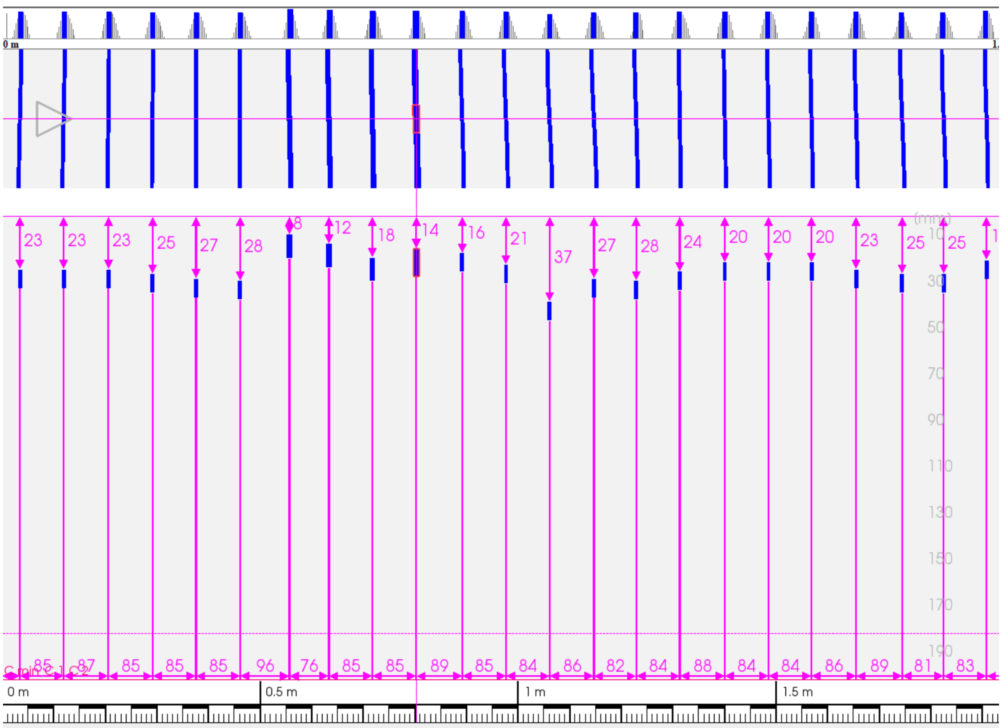


Figure 15 - Ferro scan of Langton House soffit.

- The sofit of the flats in Langton House were partially broken out to reveal the cover, spacing and size of the primary reinforcement. Site measurements indicated the diameter was 6mm bars with the spacing recorded as 90mm – 100mm. With more data recovered from the GPR and Ferro scans the spacing of the reinforcement has been taken as 90mm.

Figure 16 - Site photo of soffit break-out in Langton House

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. All information below is derived from intrusive investigation, site ferro scans and GPR scans.

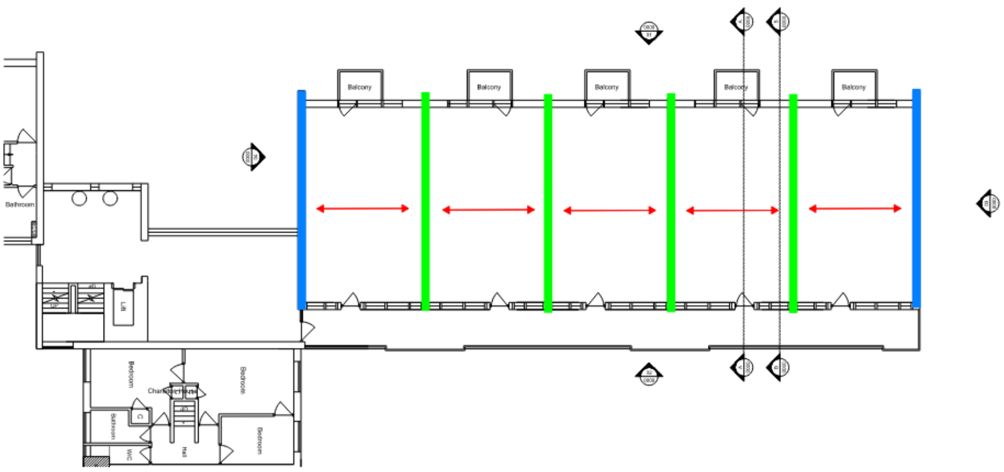


Figure 17 – Langton House typical floor layout showing "key element" locations.

The key elements, herein referred to as In-situ Flank Walls (shown in Blue), In-situ Cross Walls (Green), 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.

In-situ Flank Wall Construction (Shown in Blue)

Height: 2.465m
Construction: 178mm thick reinforced concrete load bearing wall.
Reinforcement: Two Layers 6mm square twist bars at 350mm c/c
Vertical Tie Reinforcement: 6mm (1/4") square twist bars @ 350mm c/c

In-situ Cross Wall Construction (Shown in Green)

Height: 2.465m
Construction: 152mm thick concrete loadbearing wall panel
Reinforcement: Two Layers 6mm square twist bars at 350mm c/c
Vertical Tie Reinforcement: 6mm (1/4") square twist bars @ 350mm c/c

Floor Slab Construction

Span: 5.042m (16' 6.5") max.
Construction: 195mm thick one way spanning reinforced concrete slab with 40mm screed over.
Bottom Reinforcement: 6mm (1/4") ribbed bars @ 90mm c/c
Top tie Reinforcement: 10mm (3/8") square twisted bars @ 275mm c/c, 900mm projection from wall

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:

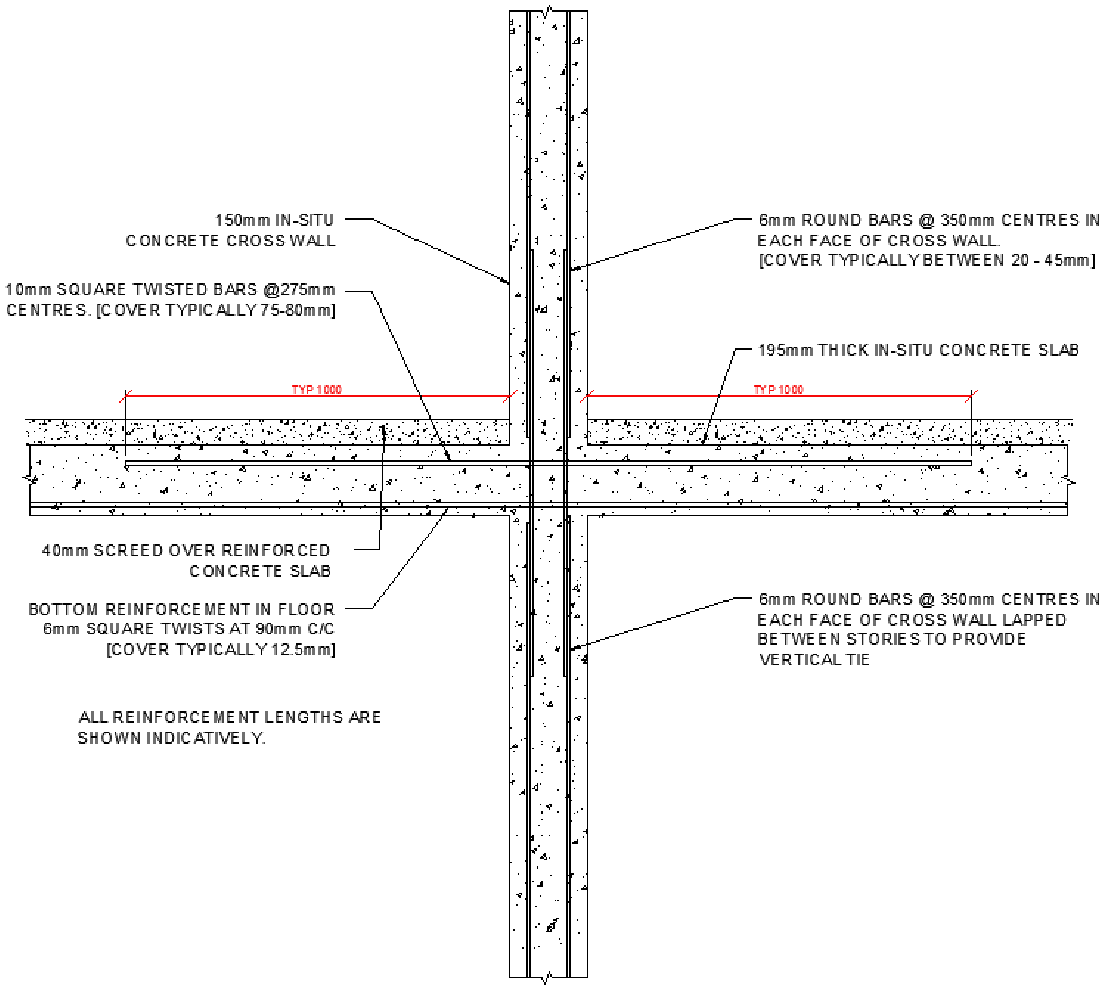


Figure 18 – Cross wall / floor slab joint.

The reinforcement within the wall panels appears to be continuous from one panel to the next with 6mm vertical bars lapped onto the primary wall reinforcement. The vertical wall reinforcement consists of two layers of 6mm square twist bars at 350mm c/c and cover ranging from 20-45mm. Horizontal ties are provided in the form of 10mm square twist bars at 275mm c/c in the top of the slab and the 6mm square twist bars in the bottom of the slab.

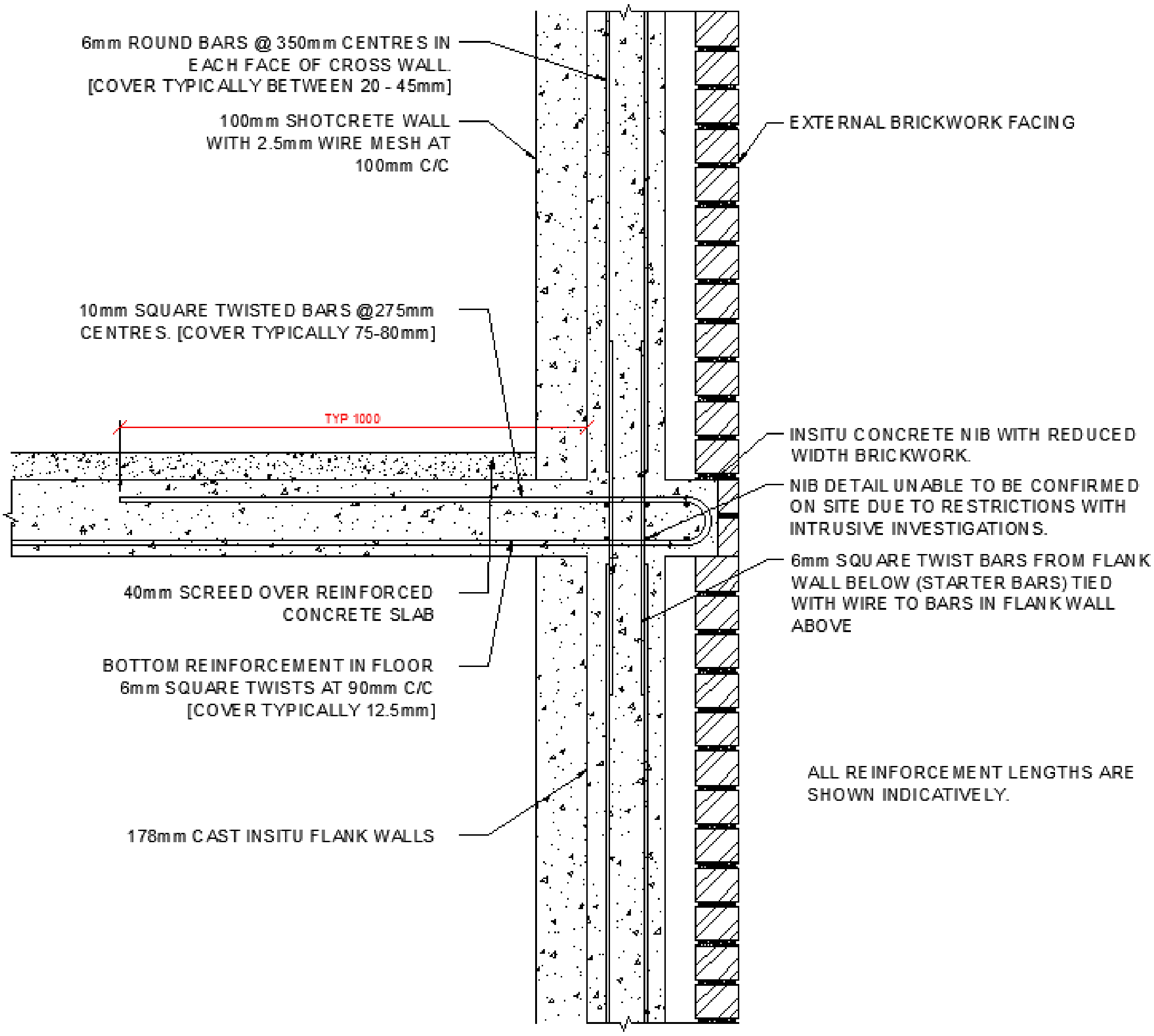


Figure 19 – Typical In-situ flank wall to floor slab joint.

The investigations into the flank walls show that the wall is in-situ reinforced concrete as highlighted in Figure 17 above. The flank walls were found to have a 100mm shotcrete liner with 2.5mm wire mesh reinforcement. 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 at 275mm c/c projecting into the wall panel. As identified in the other blocks is assumed that the bars on the back side of the wall are bent down and around 2no. 6mm and 1no. 12mm longitudinal bars providing a horizontal tie. Due to access the required opening up could not be completed in Langton House to confirm this.

Floor Construction Detail

The floor slabs are constructed from 195mm thick reinforced concrete slab which have a non-structural 40mm screed over the top. The floor slab is constructed with 10mm ($^3/_8$") ribbed bars @ 275mm c/c located in the top of the slab providing a horizontal tie between slabs, 6mm ($^1/_4$") square twist bars at 90mm c/c are located in the bottom with 12.5mm cover typically.

In the three flats surveyed it was found that there were horizontal tie bars at approximately 275mm c/c. This was found to be consistent across the building. 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.

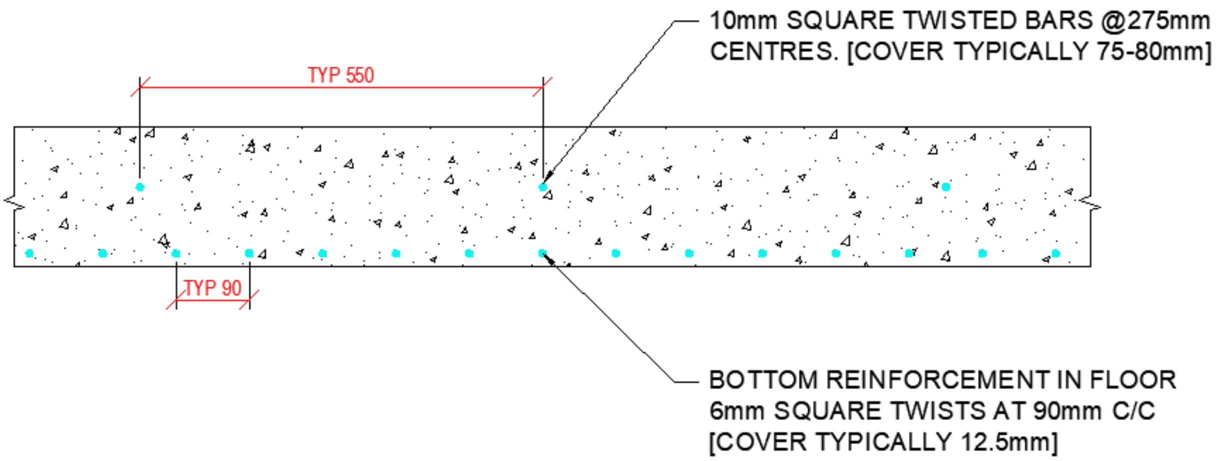


Figure 20 – Typical Langton House floor construction detail.

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.

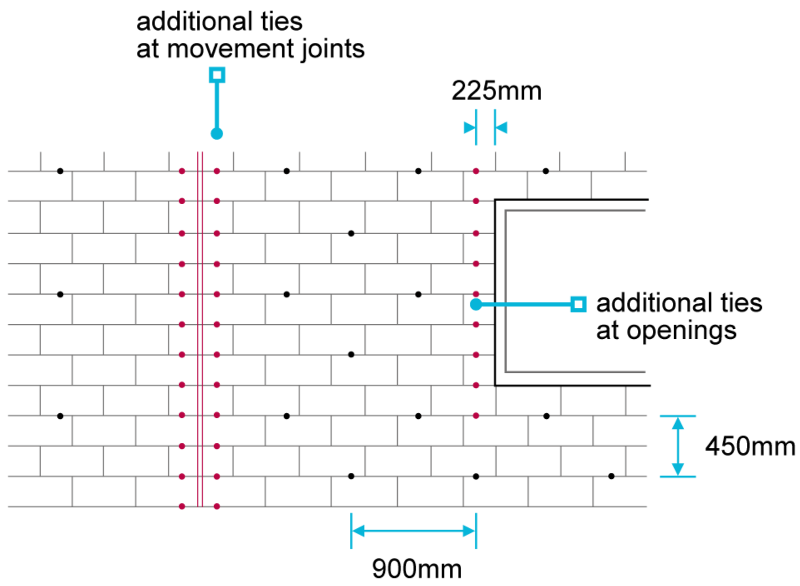


Figure 21 - Typical Wall tie detail (Ref NHBC Standard 6.1.18)

As indicated in Figure 19 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 of the masonry skins and the structural cross 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 22 - Removed blockwork showing missing or unused wall tie.

5. STRUCTURAL FIRE ASSESSMENT

Langton House is linked to Havilland house via a stair core, should Langton 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 – 12.5mm – 25mm from breaking out, and 18mm on average from ferro scans.
- Flank & Cross Wall Cover – 20mm - 45mm from breaking out (Excluding shotcrete)

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 23 - 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.

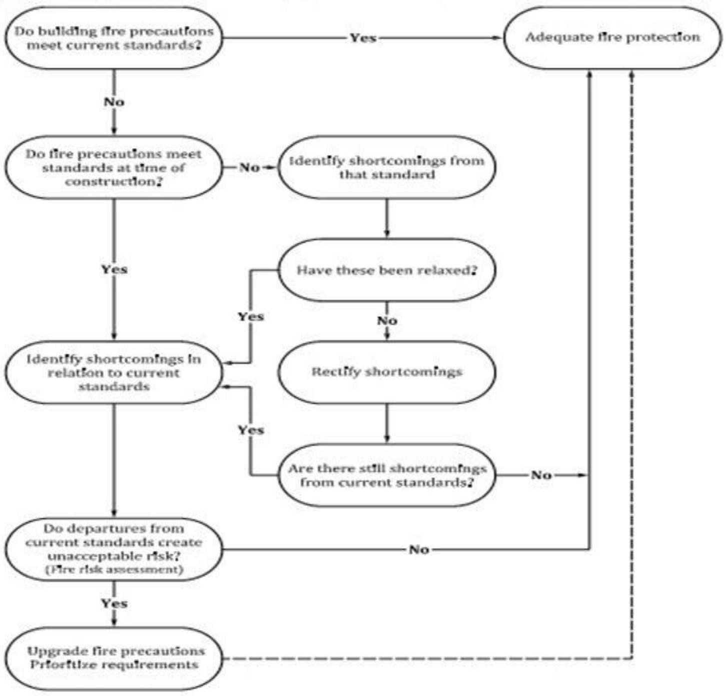


Figure 24 – Decision tree in relation to the requirement to upgrade fire precautions

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 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.

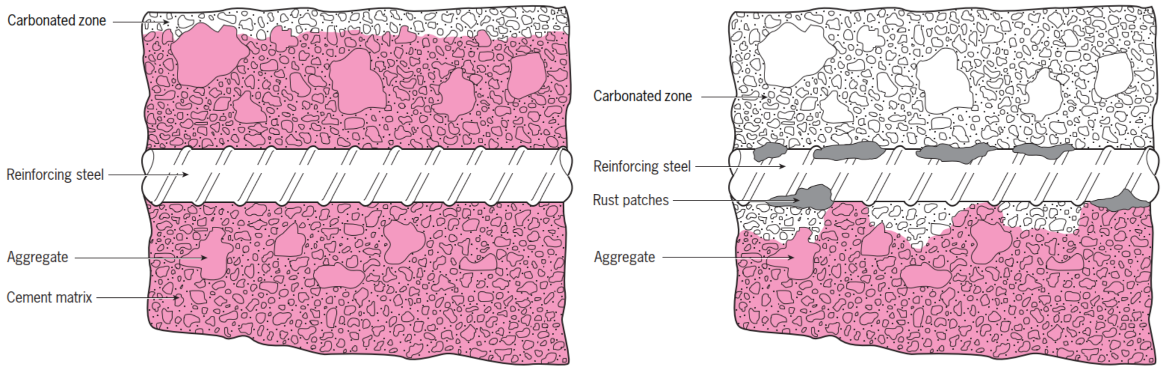


Figure 25 – (Left) Diagrammatic view of steel protected from carbonation-induced corrosion in partially carbonated concrete, (Right) Diagrammatic view of steel corroding in carbonated concrete.

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	MEMBER TYPE	CARBONATION DEPTH	CARBONATION SURPASSED REINFORCEMENT?
DS1	Slab	0-5mm	No
DS1 Wall	Wall	0-5mm	No
DS2	Slab	0-5mm	No
DS4	Slab	0-5mm	No
DS4 Wall	Wall	50-55mm	Yes
DS5 Wall	Wall	55-60mm	Yes
DS6 Wall	Wall	10-15mm	Yes
DS7	Slab	0-5mm	No
DS7 Wall	Wall	>100mm	Yes
DS7-8 Wall	Wall	>100mm	Yes
DS8 Wall	Wall	20-25mm	Yes
DS9	Slab	0-5mm	No
DS9 Wall	Wall	0-5mm	No
DS10	Slab	0-5mm	No
DS10 Wall	Wall	0-5mm	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 the flank walls, likely in an attempt to remediate the low concrete cover and fire period 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 depths in the slabs did not appear to be excessive for a structure of this age. However, there were several locations in the walls that exhibited extremely high carbonation depths. 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. 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.

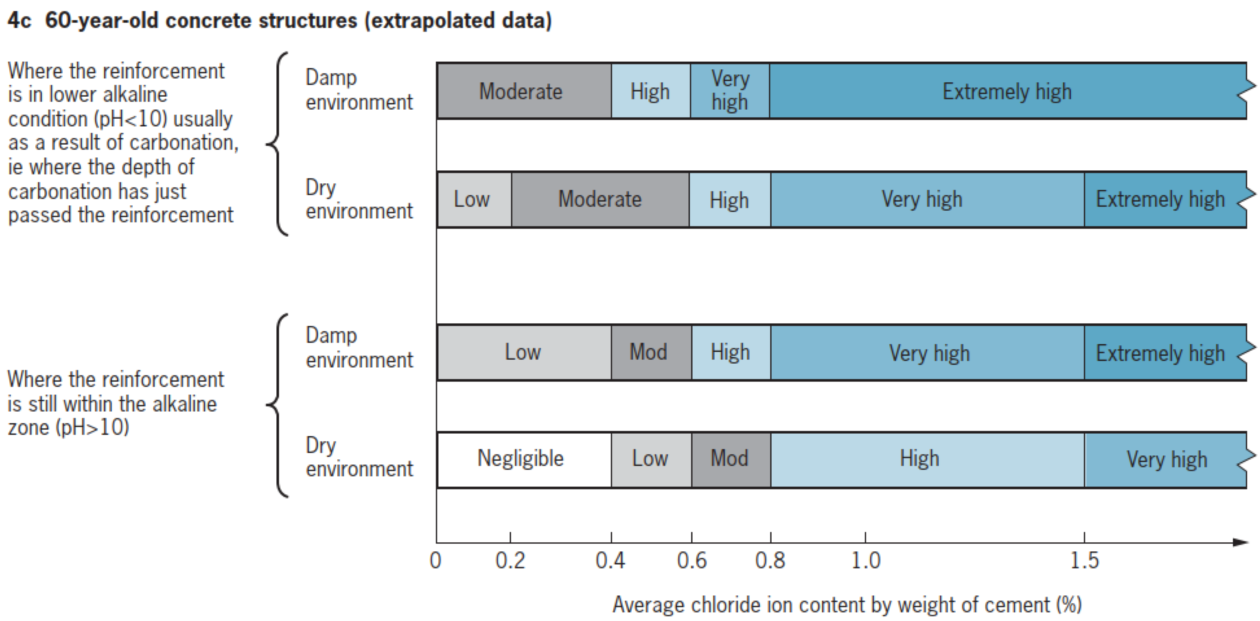


Figure 26 – Estimated risk of corrosion associated with carbonation, chloride content and environment.

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.

MEMBER TYPE	CARBONATION REACHED / SURPASSED REINFORCEMENT	CHLORIDE ION CONTENT BY MASS (%)	RISK OF STEEL REINFORCEMENT CORROSION (BRE DIGEST 444 PT1)
DS1	No	0.04	Negligible
DS1 Wall	No	0.06	Negligible
DS2	No	0.04	Negligible
DS4	No	0.18	Negligible
DS4 Wall	Yes	0.12	Low
DS5 Wall	Yes	0.08	Low
DS6 Wall	Yes	0.02	Low
DS7	No	0.06	Negligible
DS7 Wall	Yes	0.36	Moderate
DS7-8 Wall	Yes	0.10	Low
DS8 Wall	Yes	0.10	Low
DS9	No	0.04	Negligible
DS9 Wall	No	0.06	Negligible
DS10	No	0.04	Negligible
DS10 Wall	No	0.06	Negligible

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

- The chloride contents in all concrete tested was found to be low, in all locations.
- Concrete members tested were generally found to have a negligible or low risk of corrosion to the embedded reinforcement with the exception of one location with a moderate risk. 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.
- 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
Flank Wall	2.5	22.4	40.8
Cross Wall	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)
Flank Wall	6.1%	54.9%	61%
Cross Wall	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
Flank Wall	61.0 < 66.6 ∴ FAIL	9.0 > 2.0 ∴ PASS
Cross Wall	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) satisfy the expected proportions for today's standards for the slab, but do not for the walls. The concrete is therefore likely to be offering adequate protection for embedded reinforcement in the slab. However, due to the low contents of Silica (SiO_2) and Calcium Oxide (CaO) a reduced strength, increased risk of carbonation, pour workability, and decreased resistances to high temperatures can be expected of the concrete. Through core testing and hammer testing the strength of the concrete has not been found to be low, however the increased risk of carbonation can clearly be seen from the high carbonation depths identified in Table 3. The reduced resistance to heat, will also reduce the fire resistance period of the wall panels in the event of a fire.

6.4. Compressive Strength

In order to assess the robustness of the concrete elements forming Langton 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 Langton House were from cross walls and flank walls. It was not possible to undertake core sampling of floor slabs as it was not feasible to core drill into an occupied flat above or below. Where core samples were not achieved a no intrusive Schmit hammer test was carried out, the results from the hammer tests are shown in Table 8. The core samples 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 9.

Table 8 - Hammer Test results

FLAT	REF	LOCATION	COMPRESSIVE STRENGTH
Flat 11	SHT1	Soffit	42
	SHT3	Soffit	43
	SHT4	Soffit	32
	SHT5	Soffit	50
	SHT6	Soffit	52
	SHT7	Soffit	50
	SHT8	Top of Slab	50
	SHT9	Top of Slab	46
	SHT10	Top of Slab	60
	SHT11	Wall	42
	SHT12	Wall	52
	SHT13	Wall	38
Flat 17	SHT14	Soffit	47
	SHT15	Soffit	46
	SHT16	Top of Slab	56
	SHT17	Top of Slab	38
	SHT18	Top of Slab	42
	SHT19	Soffit	52
	SHT20	Wall	49
	SHT21	Wall	38

	SHT22	Soffit	32
	SHT23	Soffit	50
	SHT24	Top of Slab	43
	SHT25	Top of Slab	40
	SHT26	Wall	58
	SHT27	Wall	54
	SHT28	Wall	36
	SHT29	Wall	37
	SHT30	Wall	42
Flat 25	SHT31	Top of Slab	40
	SHT32	Top of Slab	45
	SHT33	Top of Slab	46
	SHT34	Wall	48
	SHT35	Wall	44
	SHT36	Top of Slab	38
	SHT37	Top of Slab	33
	SHT38	Top of Slab	42
	SHT39	Wall	47
	SHT40	Wall	38
	SHT41	Soffit	38
	SHT42	Soffit	44

Table 9 - Compressive strength results from the core samples taken in Langton House

REF	LOCATION	CORE LOCATION	COMPRESSIVE STRENGTH
BH1	Flat 25	Wall	52.7 N/mm^2
BH2	Flat 25	Wall	44.6 N/mm^2
BH3	Flat 25	Wall	62.8 N/mm^2
BH4	Flat 11	Wall	35.6 N/mm^2
BH5	Flat 11	Wall	30.1 N/mm^2
BH6	Flat 11	Wall	35.3 N/mm^2
BH7	Flat 11	Wall	38.4 N/mm^2
BH8	Flat 11	Wall	28.9 N/mm^2
BH9	Flat 25	Wall	51.8 N/mm^2

Using the results obtained from the laboratory testing of each core combined with the site hammer tests, 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 & hammer tests, show that the characteristic compressive strength of the concrete walls & slab in Langton House is:

Table 10 - Compressive strength results for Langton House

TEST LOCATION	COMPRESSIVE STRENGTH
Wall	**25.50N/mm^2**
Slab	**30.45N/mm^2**

The above value for the walls and slab is based on a standard deviation with the highest and lowest values removed. It may be an improved compressive strength could be determined through further core sampling.

7. LANGTON STRUCTURAL ASSESSMENT

7.1. Assessment Criteria

The Langton House block has been assessed against the Approved Document A requirements for structural design and detailing for a consequence class 2b building requiring the provision of horizontal and vertical ties as defined in section 5.2 & 5.3 of the approved document. The following sections document the main findings of the investigation and a summary of each criterion of the assessment.

7.2. Criterion 1 & 2– Adequation Provision of Horizontal & Vertical Ties

The first stage in the assessment to determine the adequacy of the joints based on 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> Sufficient
<u>Vertical Ties:</u> Sufficient

Ties have been installed at regular spacing across joint locations. Generally, ties were located at 275mm c/c for horizontal ties and 250mm - 350mm c/c for vertical ties. Through analysis it has been concluded that the existing provision of horizontal ties does meet the requirements for the resisting the minimum tie force.

The cross-wall joint is therefore **sufficient** to pass the assessment for a Consequence Class 2b building.

Flank Wall / Floor Slab Joints

The assessment of the Flank wall / floor slab joint has shown that:
<u>Horizontal Ties:</u> Sufficient

Vertical Ties: Sufficient

The walls have 10mm square twisted bars cast into the walls at circa 275mm 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.

Table 11 – Langton House tie details summary table

Langton House (Consequence Class 2B)			
Joint Type	**Adequate horizontal tie**	**Adequate Vertical Tie**	**Notes**
Flank Wall	a	a	Sufficient
Cross Wall	a	a	Sufficient

Langton House – Criterion 1 & 2 – Adequate Provision of Ties
Sufficient provision of ties

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. This indicates that the concrete externally is carbonated and should be monitored.
- 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 tests completed on the slab internally gave a gave a negligible or low risk of corrosion to the embedded reinforcement within the slab elements. The testing of the wall elements showed a low to high risk of corrosion for the wall elements, as indicated by the spalling concrete noted externally. 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, cement composition and levels of carbonation. Testing of the external walls was only completed internally and further carbonation tests are recommended on the outside of the block to determine the carbonation depth and the possible impacts this has on the spalling identified.
- The concrete walls and floor slabs have a characteristic compressive strength of 25.20N/mm², and 30.45N/mm², respectively. [Note this is based on core sampling of the wall elements only. The floor slabs have been assessed with a hammer test 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

- Horizontal & Vertical ties between cross walls and flank walls were observed, with an adequate tie being provided across the building.
- Cast in L-bars have been assumed based on the other blocks at St Judes, to provide a horizontal tie into the flank walls. The L-bars link around a 12mm lacer bar in the flank wall and extend 900mm into the floor slabs.
- A 100mm shotcrete remedial detail has been applied to the flank walls across the building. Internal walls have a plaster finish applied directly to the concrete wall.
- 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 Langton House in its current state is adequate to resist disproportionate collapse due to the adequate provision of vertical & horizontal ties between elements.

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 proximity and shared lift and stair core with Havilland House. The current slab achieves an estimated R60 fire period. 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.

Visual observations and non-intrusive investigations 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 fire loading assessment and condition of the concrete to reduce the risk to as low as reasonably possible.

As the stair cores are linked between the blocks as noted above, the recommendations in relation to fire, controls and risk reduction measures for Haviland have been carried across where deemed applicable:

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. Full condition survey of the balustrades around Langton House, temporary support provided to those in a critical condition with a design and programme developed to replace all the balustrades.
5. 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.
6. 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. Repairs and or replacement of the residential balconies due to deterioration from carbonation.
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:

- 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.

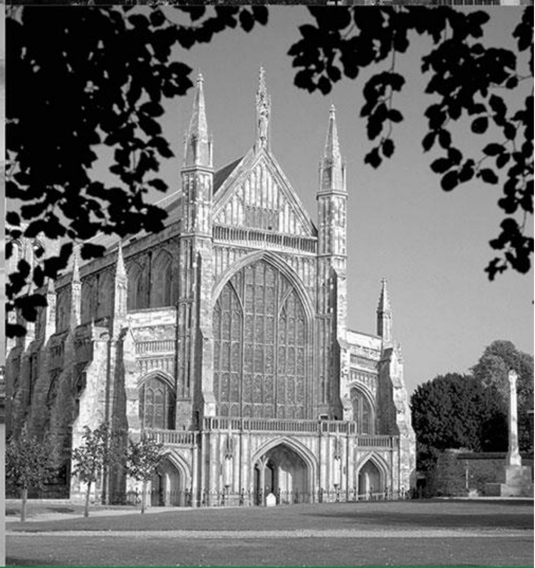
- Visual inspections at 10-year intervals to structural joints which are vulnerable to water penetration; locations such as flank walls and roofs.

- 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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