RIDGE

SHROPSHIRE HOUSE
STRUCTURAL ROBUSTNESS
ASSESSMENT REPORT
LONDON BOROUGH OF ENFIELD
June 2022

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ASSESSMENT REPORT
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1. INTRODUCTION

1.1. Site Address

Shropshire House
Cavendish Close
Enfield
London
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1.2. Structural Engineering Brief

Ridge and Partners LLP (Ridge) were appointed by the London Borough of Enfield to carry out structural investigations to determine the robustness of the dwelling block, Shropshire House, Enfield, London. The appointment came following owners of LPS dwelling blocks, including London Borough of Enfield, were advised to seek professional advice regarding the safety of their assets by the Ministry of Housing, Communities & Local Government (MHCLG).

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

1.3. Report Contents

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

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

1.4. Limitations

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

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

All flats within the block are single level (no duplex apartments). It was therefore not possible to obtain core samples from floor slabs during the investigative phase. Ridge, on their final visit to the block, noted that core sampling could be undertaken within one area in the communal stairwell which would reduce the impact on residents from this intrusive works. However, this area had a textured ceiling coating and as such was believed to be containing asbestos. We advised the client to undertake asbestos testing / removal on this element so that this could be undertaken in a follow-up visit, although this has yet to be undertaken.

1.5. Statement

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

Figure 1 – Shropshire House Location (Google Maps, 2022)

Figure 2 – Shropshire House (Google Maps, 2022)

2. EXECUTIVE SUMMARY

The Large Panel System (LPS) dwelling block, Shropshire House, Enfield, London has been assessed for its robustness to resist accidental loading and its susceptibility to progressive collapse.

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

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

LPS Criterion 1 – Adequate Ties (Reinforcement) within Joints

SHROPSHIRE HOUSE – ADEQUATE TIES	
JOINT	PASS / FAIL
Flank Wall	√ *
Cross Wall	X
Walls to Wall Joints	X
CONCLUSION - FAIL	

* Flank corner panel found to fail due to no mechanical ties, only main panels pass assessment.

LPS Criterion 2 – Adequate Strength to Resist Accidental Loads

Shropshire House, in its current state, has a piped-gas supply serving the properties. The BRE Report 511 states that the overpressure from an internal explosion of piped-gas is 34kN/m^2. The below table shows the outcome of the checks on each structural member for the above loading.

SHROPSHIRE HOUSE – KEY ELEMENT CHECKS (WITH PIPED-GAS SUPPLY – 34KN/M^2)	
JOINT	PASS / FAIL
Flank Wall	X
Cross Wall	X
Floor Slabs	X
CONCLUSION - FAIL	

LPS Criterion 3 – Ability to Mobilise Alternative Load Paths

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

SHROPSHIRE HOUSE
Unable to mobilise alternative load paths
CONCLUSION - FAIL

The conclusion is therefore that the blocks in their current state are inadequately robust to prevent disproportionate collapse in the event of an internal gas explosion, due to the piped-gas supply.

LPS Criterion 2 – Adequate Strength to Resist Accidental Loads – Re-assessment with the Removal of the Piped-Gas Supply

The BRE Report 511 states that the overpressure from an internal gas explosion in a block without a piped-gas supply can be reduced to 17kN/m^2 to allow for explosion from other sources such as gas canisters. The below table shows the outcome of the reassessment of each structural member for the reduced loading, should it be decided to remove the piped-gas supply.

SHROPSHIRE HOUSE – KEY ELEMENT CHECKS (WITHOUT PIPED-GAS SUPPLY – 17KN/M^2)	
JOINT	PASS / FAIL
Flank Wall	√
Cross Wall	X *
Floor Slabs	X
CONCLUSION - FAIL	

*Cross walls found to fail in the upper 8 floors (lower floors pass due to increased compression)

LPS Criterion 3 – Ability to Mobilise Alternative Load Paths – Re-assessment with the Removal of the Piped-Gas Supply

As the joints between members are inadequately tied and all main structural members were shown to be inadequately robust to resist even the reduced overpressure, it is unlikely that alternative load paths could be mobilised in the event of an internal explosion.

5-STOREY BLOCKS
Unable to mobilise alternative load paths
CONCLUSION - FAIL

The conclusion is that, regardless of whether the piped-gas supply is removed, Shropshire House fails all of the assessment criterion and is therefore considered to be inadequately robust to resist disproportionate collapse.

<u>Our recommendations are as follows:</u>
To address the failings of the disproportionate collapse requirements, works are required to the block. It is likely that, if the blocks are to be retained long-term, that this will include strengthening works.
As a short-term, risk-reduction measure the piped-gas supply should be removed in a phased approach. This process should be commenced as soon as practicably possible.
The piped-gas supply should be replaced by an alternative source.
The required remaining life of the block should be discussed.
A risk analysis should be carried out to determine:

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

If the risk analysis shows it to be required, a suitably qualified structural engineer should carry out strengthening proposals for the blocks.
A cost-benefit analysis should then be carried out, accounting for the short remaining life of the blocks, to understand whether the strengthening works are suitable.
Accelerated demolition programme may need to be considered following the results of the cost-benefit analysis.
Further information regarding the recommendations can be found in Section 8.

3. BRIEF HISTORY OF LPS BLOCKS AND DISPROPORTIONATE COLLAPSE

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

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

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

4. SHROPSHIRE HOUSE INVESTIGATION

4.1. Investigation Overview

The dwelling block, Shropshire House located in Enfield, London has been assessed for its robustness to resist accidental loading from over-pressure, such as an internal gas explosion, and its susceptibility to progressive collapse. The block is believed to have been constructed by Direct Municipal Labour from a Large Panel System (LPS) for the London Borough of Enfield in the late 1960's / early 1970's. (Committee Approval Date: 1968, Construction Completion Date: 1971)

4.2. Investigation Methodology

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

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

4.3. Main Findings of the Investigation

Figure 3 – Typical floor plan for sister block, Walbrook House (no full floor plans available for Shropshire House)

Floor Slab Construction

Span: 3.75m (max.)
Construction: 200mm thick, solid concrete floor slab [To be confirmed by core sampling]
Bottom Reinforcement: 12mm square twisted bars, 300mm c/c + 12mm diameter U-bars @ slab edge
Top Reinforcement: Unreinforced

Flank Wall Construction

Height: 2.3m
Construction: ~300mm thick concrete loadbearing wall panel (no outer leaf or insulation found in core samples)
Reinforcement: Internal face: 2no 30mm diameter dowel bars @ 340mm c/c + 1no intermediate 24mm square twisted bar between dowels (see details)
External face: Unknown – too deep to scan, but believed unreinforced

Cross Wall Construction

Height: 2.3m
Construction: 175mm thick concrete loadbearing wall panel
Reinforcement: Effectively unreinforced (provisions for lifting only)

Spine Wall Construction

As per Cross Wall

Joint Construction

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

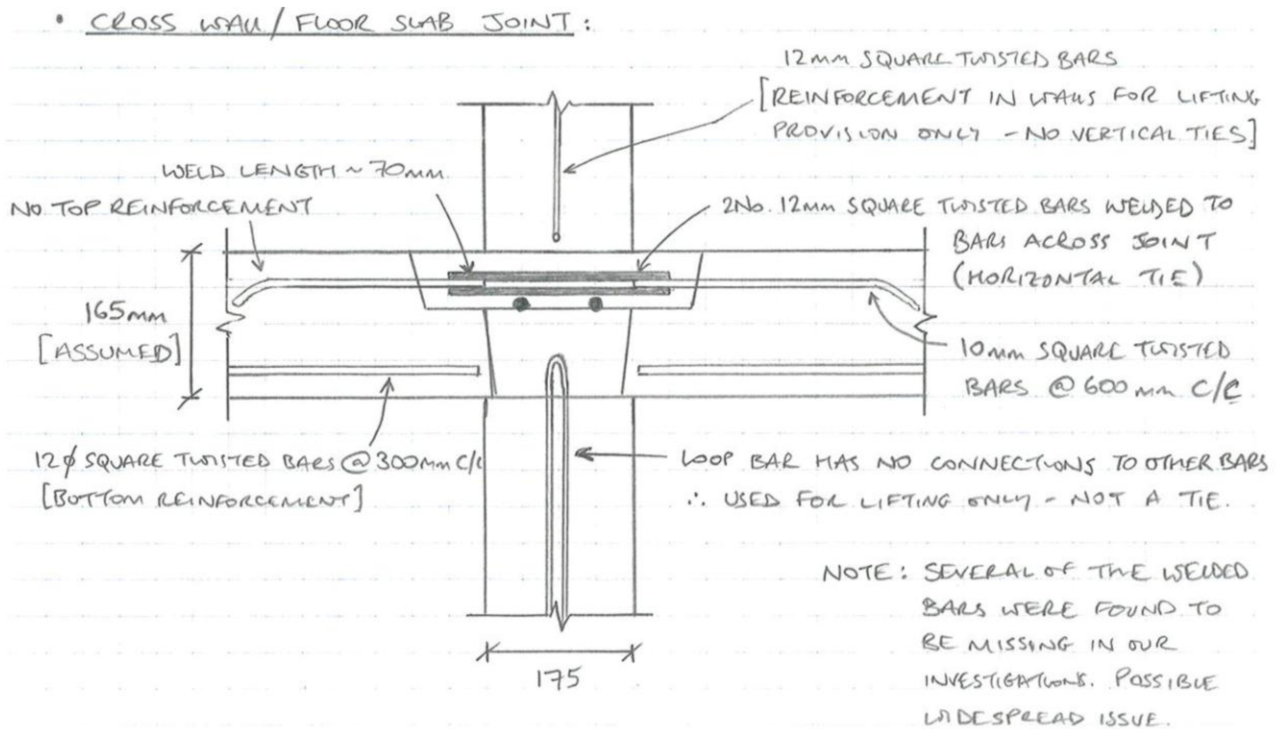


Figure 4 – Cross wall / floor slab joint

*We noted that the centres between the horizontal tie details was different in Flat 153 than in Flat 61. Flat 153, shown above, had ties set at 600mm c/c whereas Flat 61 these were found at 400mm c/c. The worst case (600mm c/c) has been adopted in our assessment.
*Also at least one horizontal tie in cross wall joint found to be defective, with no welded bars to form the connection.

• CROSS WALL (STAIRWELL) JOINT:

APPEARS TO FOLLOW SAME PRINCIPAL AS STANDARD CROSS WALL DETAIL, EXCEPT THERE IS NO FLOOR SLAB OTHER SIDE OF WALL SO WELDED CONNECTION IS REDUNDANT AS NOTHING TO WELD TO ON STAIRWELL SIDE.

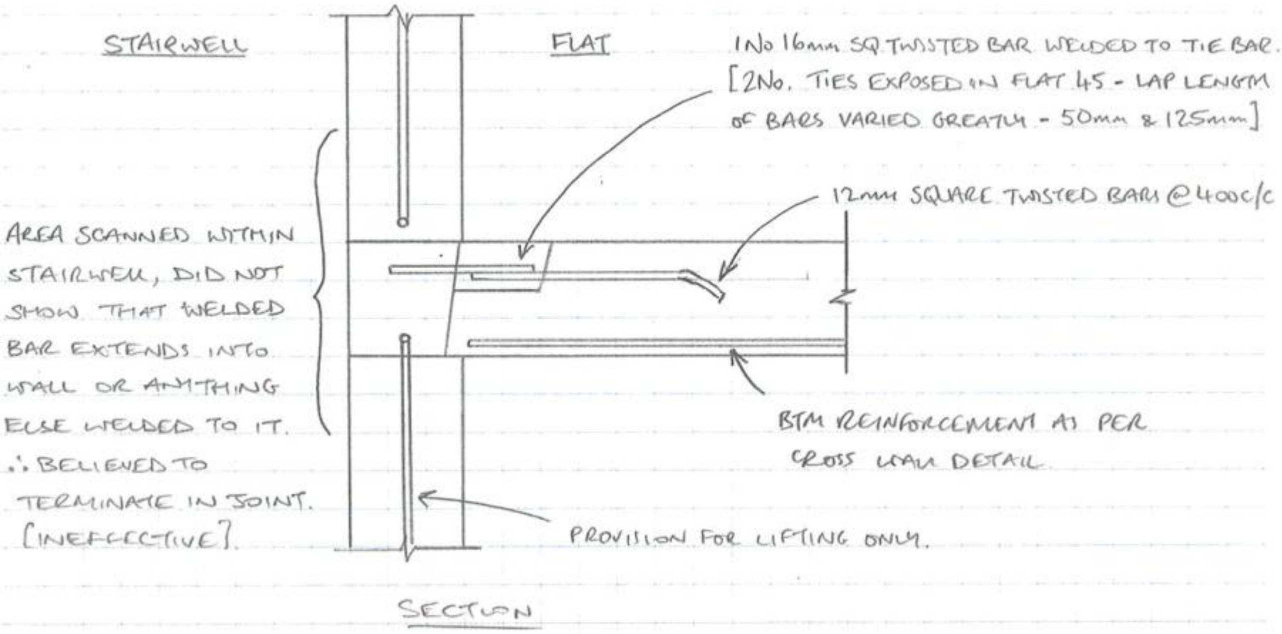


Figure 5 – Cross wall / floor slab joint (stairwell)

• FLANK WALL / FLOOR SLAB JOINT:

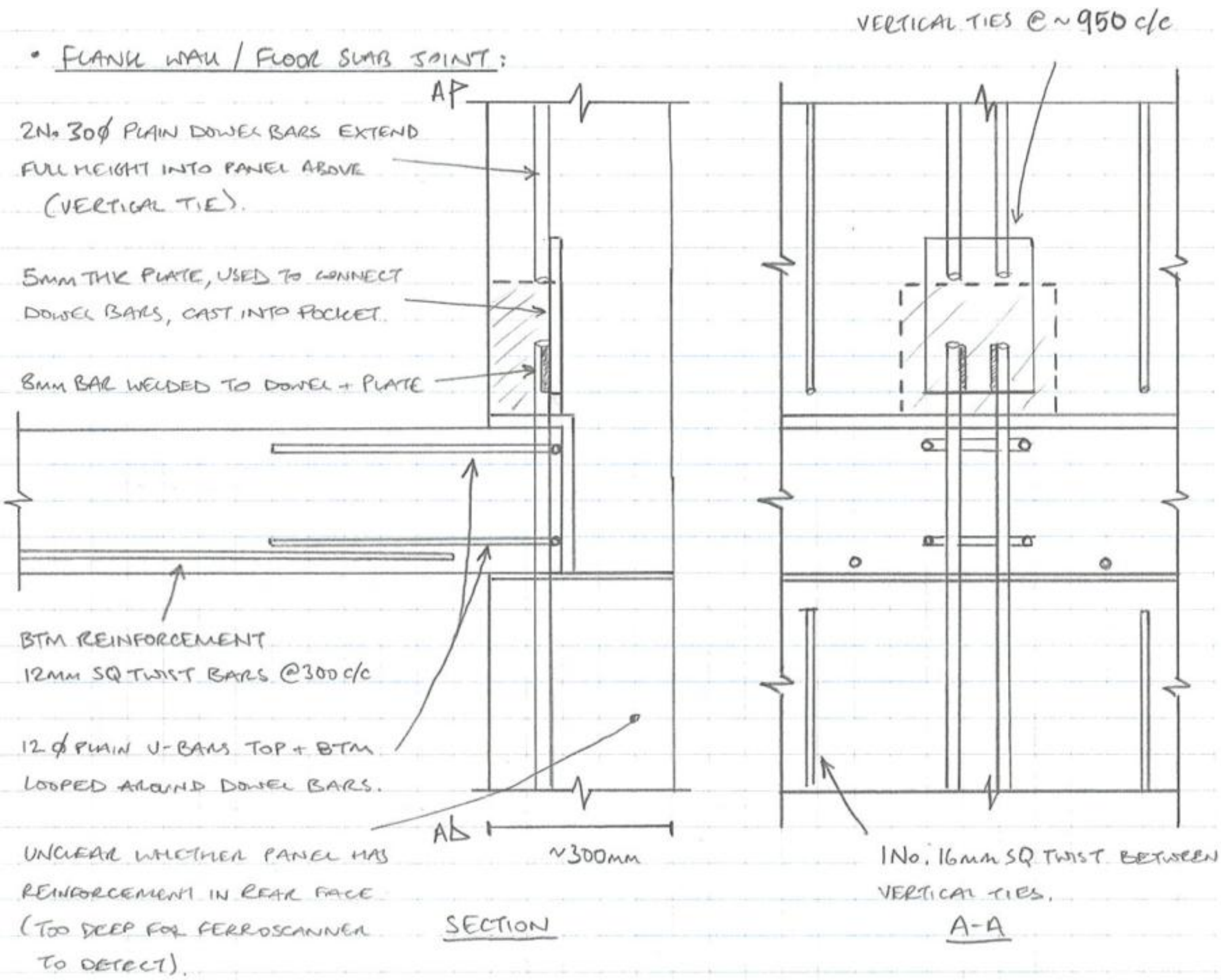


Figure 6 – Flank wall / floor slab joint

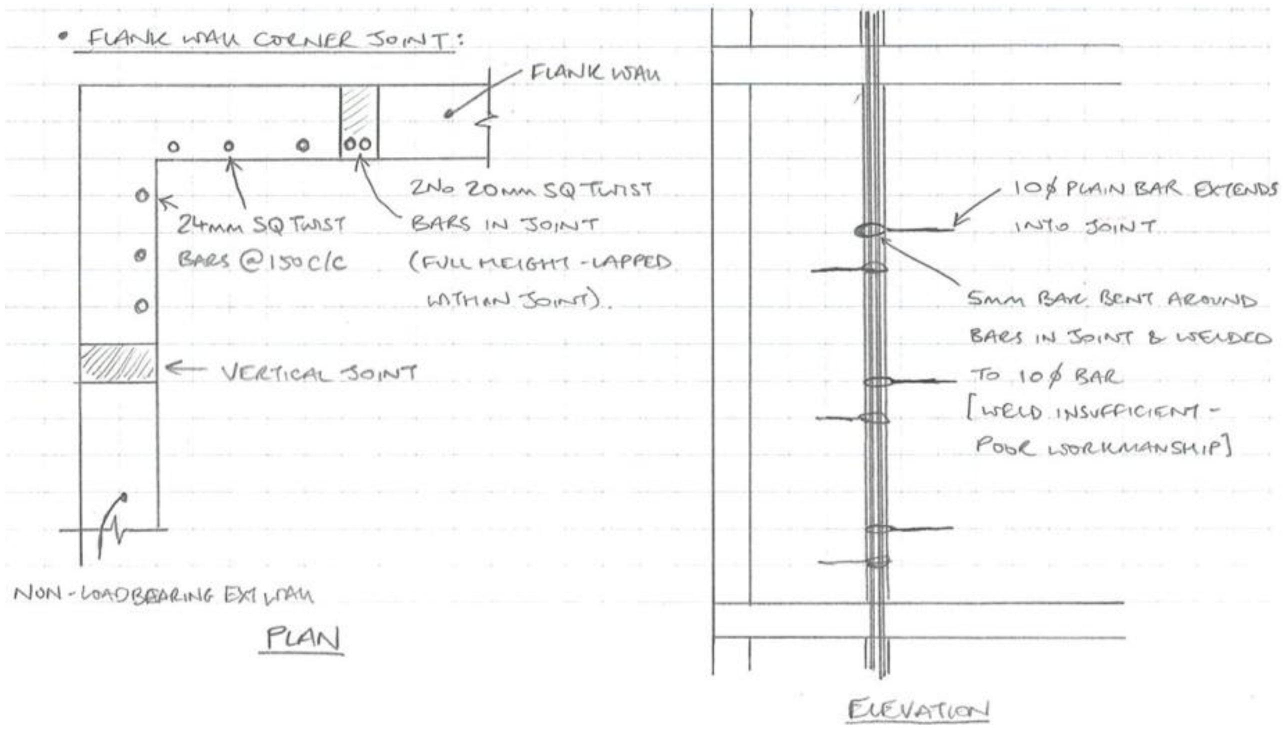


Figure 7 – Flank wall corner panel

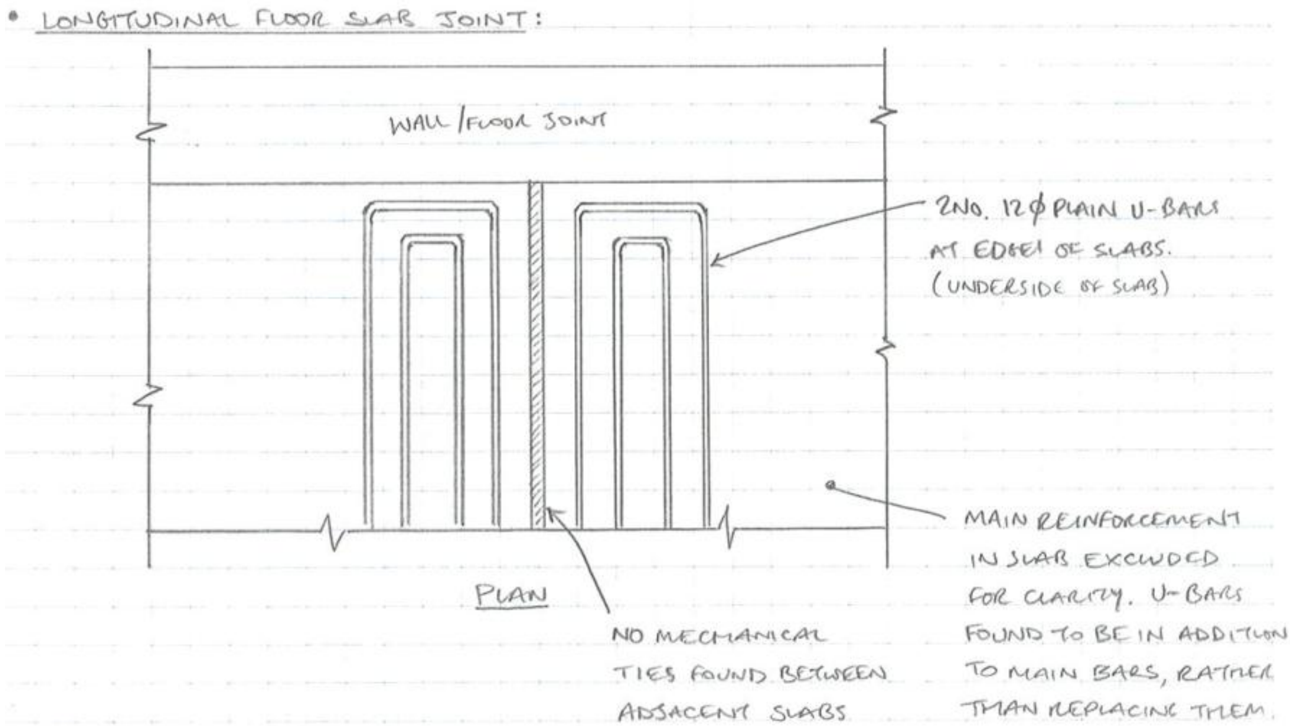


Figure 8 – Longitudinal floor slab joint

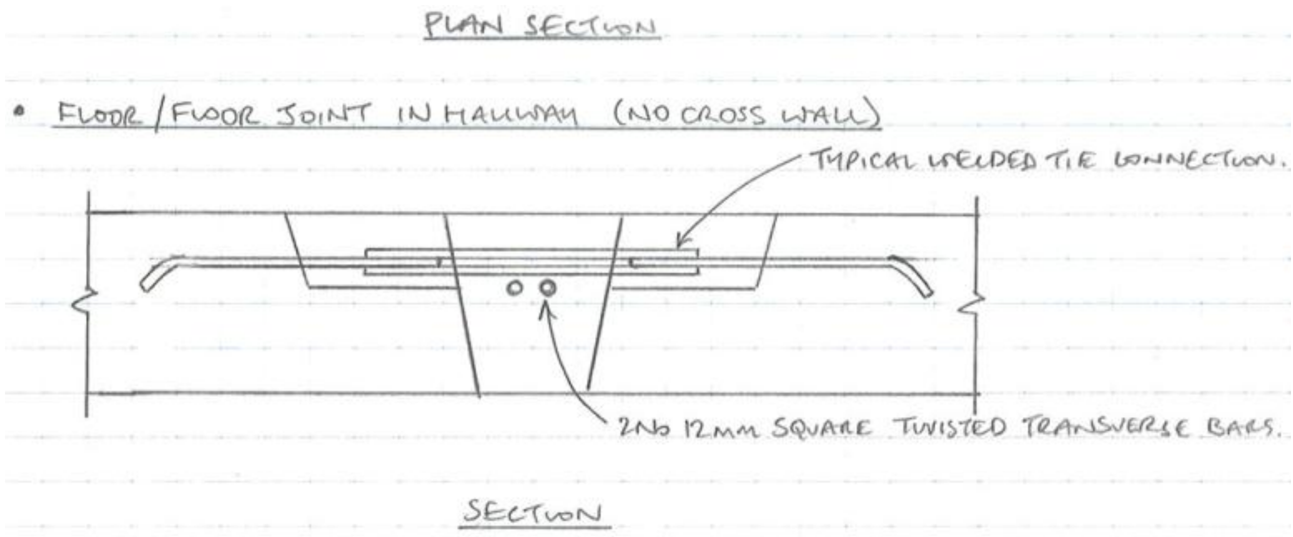


Figure 9 – Floor to floor joint in hallway (no cross wall)

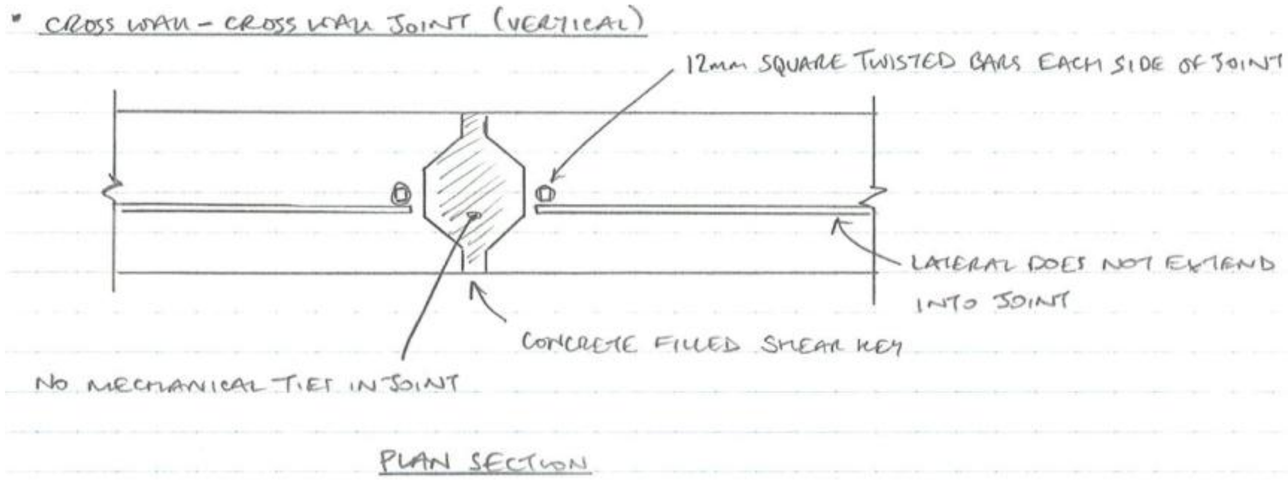


Figure 10 - Cross wall / cross wall joint

4.4. Concrete Testing

Carbonation

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

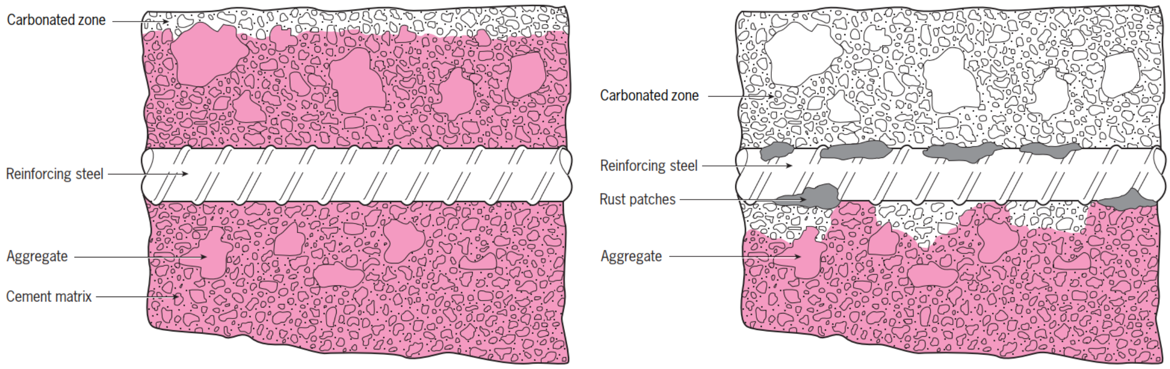


Figure 11 – (Left) Diagrammatic view of steel protected from carbonation-induced corrosion in partially carbonated concrete, (Right) Diagrammatic view of steel corroding in carbonated concrete. (BRE, 2000)

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

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

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

CARBONATION TEST RESULTS					
Flat	**Test Location**	**Member Type**	**Carbonation Depth**	**Min. Cover to Bar**	**Carbonation Surpassed Reinforcement?**
Flat 45	1	Floor Slab	25mm	11mm	Yes
Flat 45	2	Flank Wall	5mm	19mm	No
Flat 45	3	Cross Wall	20mm	45mm	No
Flat 61	4	Floor Slab	12mm	20mm	Yes *
Flat 61	5	Cross Wall	8mm	[Central]	No
Flat 61	6	Cross Wall	5mm	[Central]	No
Flat 153	7	Flank Wall	12mm	25mm	No
Flat 153	8	Cross Wall 1	3mm	[Central]	No
Flat 153	9	Cross Wall 2	11mm	[Central]	No

Table 1 - Carbonation Depths

In the majority of the nine locations, the carbonation depth was observed to be relatively shallow and had not surpassed the depth of the embedded reinforcement, indicating that the rebar remains within a passive environment. However, the floor slab within Flat 45 was shown to have a carbonation depth exceeding the reinforcement depth, and the floor slab in Flat 61 (due to the high chloride content, see Table 2) will also likely have carbonated to a depth exceeding the cover to the reinforcement.

Chlorides

Chloride testing was carried out by drilling the concrete with a hammer drill and the dust created collected and transferred into sealable bags. 9 no. dust samples were collected from across the three flats to be tested. The samples were then sent to Sandberg LLP's Clapham laboratory to conduct laboratory testing. The site is a UKAS accredited testing laboratory No. 0262.

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

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

The BRE have published a series of diagrams in Digest 444 part 2 which can be used as a part of the assessment of chloride levels in concrete members, for 25, 40 & 60 year old structures. The diagrams show the risk of reinforcement corrosion within concrete elements for the given conditions for the respective age groups. The building had been completed in 1971, meaning the property is roughly 51 years old at the time of inspection. The concrete testing results will therefore be compared against the BRE 444 diagram for a 60 year old structure, as this best represents the structure. This diagram is shown in Figure 12.

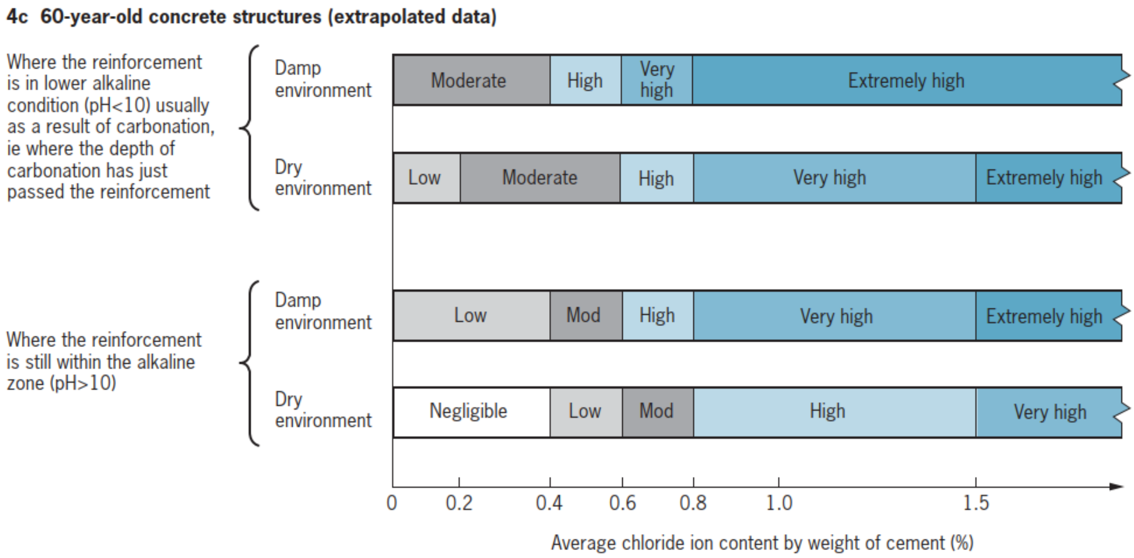


Figure 12 – Estimated Risk of Steel Reinforcement Corrosion Associated with Carbonation, Cast-in Chloride Content and Environment Conditions (BRE, 2000)

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

CHLORIDE TESTING						
	Test Location	**Member Type**	**Carbonation Reached / Surpassed Reinforcement**	**Atmosphere**	**Chloride Cl % by Mass of Cement**	**Risk of Steel Reinforcement Corrosion (BRE Digest 444 pt1)**
Flat 45	1	Floor Slab	Yes	Dry	0.76	High
Flat 45	2	Flank Wall	No	Dry	0.35	Negligible
Flat 45	3	Cross Wall	No	Dry	0.66	Moderate
Flat 61	4	Floor Slab	Yes *	Dry	0.68	High
Flat 61	5	Cross Wall 1	No	Dry	0.53	Low
Flat 61	6	Cross Wall 2	No	Dry	0.81	High
Flat 153	7	Flank Wall	No	Dry	<0.02	Negligible
Flat 153	8	Cross Wall 1	No	Dry	0.37	Negligible
Flat 153	9	Cross Wall 2	No	Dry	<0.02	Negligible

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

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

- Both of the floor slabs tested were found to be at high risk of corrosion due to the reinforcement lying within the carbonated zone (therefore the concrete is no longer offering a protective environment to the embedded reinforcement) and the high concentration of chlorides.
- All other members were noted to have reinforcement within the un-carbonated zones and therefore offered protection from the alkalinity of the concrete. The increased risk of corrosion of some members

tested is therefore related to the concentration of chlorides. The Cross Wall 2 in Flat 61 was found to be considerably high chloride content and is therefore also at high risk of corrosion to the embedded reinforcement, despite the depth of the reinforcement.

- As all members that were tested were internal, and the block is not in a coastal area, it is believed the high chloride content in the concrete members may be attributed to admixtures in the concrete mix. It is unlikely this degree of chloride concentration has ingressed over the life of the structure.
- Four of the nine internal concrete elements tested were found to be at negligible risk of corrosion to the embedded reinforcement.

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

Cement Composition

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

The full results of the chemical analysis to determine the chloride content can be seen in Appendix B, with an extract shown in Table 3. The results of the chemical analysis were then interpreted to understand the percentage weight of each chemical component against the total weight of the binder, shown in Table 4. This was then compared to the requirements from BS EN 197-1:2011 – *"Cement. Composition, specification and conformity criteria for common cements"* as a guide to determine whether the cement composition would be acceptable to today's standards, shown in Table 5.

BS EN 197-1:2011, Section 5.2.1 states that "Portland cement clinker is a hydraulic material which shall consist of at least two-thirds by mass of calcium silicates ($3CaO \cdot SiO_2$ 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." (BSI, 2011).

CEMENT COMPOSITION RESULTS				
% by weight of sample (from Appendix A)				
Flat	**Test Member**	**SiO_2**	**CaO**	**Total Cement Content**
Flat 153	Cross Wall 1	3.68	10.41	16.1

Table 3 - Sandberg cement composition test results

CEMENT COMPOSITION INTERPRETATION				
% by weight of cement (from Interpretation)				
Flat	**Test Member**	**SiO_2**	**CaO**	**Total (SiO_2 + CaO)**
Flat 153	Cross Wall 1	22.9	64.7	87.6

Table 4 - Interpretation of Sandberg Cement Composition Testing

CEMENT COMPOSITION COMPLIANCE WITH BS EN 197-1:2011 (5.2.1)		
Test Member	**Cement Consists of at least 2/3 ($CaO + SiO_2$)**	**The Ratio of $CaO / SiO_2 > 2.0$**
Cross Wall 1	87.6 > 66.6 ∴ PASS	2.83 > 2.0 ∴ PASS

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

By inspection of the interpreted results, the quantities of Silica (SiO_2) and Calcium Oxide (CaO) satisfy the expected proportions for today's standards.

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

Compressive Strength

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

FLAT	SAMPLE	CORE LOCATION	COMPRESSIVE STRENGTH
Flat 45	1	Cross Wall 1	43.9 N/mm^2
Flat 45	2	Cross Wall 2	39.8 N/mm^2
Flat 45	3	Flank Wall	72.5 N/mm^2
Flat 61	4	Cross Wall 1	34.1 N/mm^2
Flat 61	5	Cross Wall 2	37.1 N/mm^2
Flat 153	6	Cross Wall 1	36.0 N/mm^2
Flat 153	7	Cross Wall 2	34.6 N/mm^2
Flat 153	8	Flank Wall	72.0 N/mm^2

Figure 13 – Compressive Strength results from the core samples taken in Shropshire House

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

However, it should be noted that the results have a large degree of variation between the cross walls and the flank walls (both flank wall cores tested to be significantly higher at ~72 N/mm^2). This may suggest that the flank walls were designed to have a higher concrete strength than the cross walls. Typically, in LPS blocks we have found that all wall panels (if not floor slabs as well) are all cast with similar concrete compressive strengths (although there usually is some discrepancy due to poor quality control during casting and compression hardening over the years). During the intrusive phase, based on the assumption all walls are typically cast with similar concrete grades, Ridge obtained only 2 flank wall samples (cross walls are more abundant within the block, and flat 61 contained no flank walls). It is therefore not possible to conduct a separate assessment on the flank wall results using the codes noted above due to the sample size. In the first instance, the assessment was therefore carried out ignoring the flank wall results and conservatively adopting the calculated characteristic compressive strength of the cross walls for all loadbearing members, with the provision that if the latter disproportionate collapse assessment found only the flank walls to fail due to this reduced compressive strength, that further core testing could be carried out at a later date (ultimately found not to be required).

The calculations, based on the cross wall core results, show that the characteristic compressive strength of the concrete at Shropshire House is **30.1 N/mm^2**.

'Slight honeycombing' in the concrete was observed in the testing of core samples 1, 2 and 5.

4.5. Observations during the Intrusive Phase

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

- There appeared to be poor quality control during the casting of the floor slab. The concrete cover to the embedded reinforcement appeared to vary considerably throughout the areas inspected. In some areas the cover to the reinforcement was noted as <10mm. This will have consequences in terms of both risk of corrosion, but may also present a risk to fire resistance. We would advise Enfield review this with a suitably qualified Fire Risk Assessor.
- Two horizontal ties (in different flats) in the cross wall joints were found to be defective. In one of the flats the bar was found to be in a pocket surrounded by polystyrene, rather than concrete (a thin layer of concrete was on top of the polystyrene). The bar was therefore not protected by the concrete and the end of the bar had subsequently suffered from corrosion, with a small degree of sectional loss. The bar was also noted to be missing the two welded bars which pass through the cross wall, rendering this redundant and would place the surrounding ties under increased stresses in the event of an accidental event. It is unclear whether this is a wider issue throughout the block.
- In one of the joints in the cross wall / floor joint a mass of rope was found. This had created a void in the insitu concrete joint and the bars around this were found to be suffering from surface corrosion and a small degree of sectional loss.
- Despite the corrosion noted to some of the embedded reinforcement, no areas of spalling were noted within the properties inspected, on in the communal areas.

5. SHROPSHIRE HOUSE STRUCTURAL ASSESSMENT

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

5.1. Assessment Criteria

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

- LPS Criterion 1. There is adequate provision of horizontal and vertical ties to comply with the current requirements for the relevant Consequence Class for each block as set down in the codes and standards quoted in Approved Document A – Structure as meeting the requirement set down in the Building Regulations.

- LPS Criterion 2. An adequate collapse resistance can be demonstrated for the foreseeable accidental loads and actions.
  - The blocks are currently fitted with a piped-gas supply, and as such the main structural members are to be assessed for an overpressure of $34kN/m^2$.
  - Should the structure be shown to be inadequate for the above loading, the assessment shall be repeated for a reduced overpressure of $17kN/m^2$ – this would be associated with a block without a piped-gas supply.

- LPS Criterion 3. Alternative paths of support can be mobilised to carry the load, assuming the removal of a critical section of the load bearing wall in the manner defined for Class 2b in Approved Document A – Structure or alternatively assuming the removal of adjacent floor slabs (taking the floor slabs bearing on one side of the wall at a time) providing lateral stability to the critical section of the load bearing wall being considered. (BRE, 2012)

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

5.2. LPS Criterion 1 – Adequation Provision of Ties

The first stage in the assessment to determine the adequacy of the joints is to define the 'Consequence Class' of the blocks. Based on the definitions provided Building Regulations Approved Document A the block falls into Consequence Classes 3. The block therefore requires effective horizontal and vertical ties. The details for the joints between floors and walls can be seen in Section 4.3.

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

Cross Wall / Floor Slab Joints

The assessment of the cross wall / floor slab joint has shown that:
<u>Horizontal Ties:</u> Effective *
<u>Vertical Ties:</u> None

The cross wall joint is therefore **<u>insufficient</u>** to pass the assessment for a Consequence Class 3 building.

* However, we would note that at least one horizontal tie was found to be defective during our investigation. We found that the 10mm bar protruding from the slab did not have any bars welded to it to form the connection (the bar was also suffering from surface corrosion). If this is a wide spread issue in the block, it is possible that the effectiveness of the ties will the ties adequately constructed will not be sufficient to prevent disproportionate collapse.

Cross Wall / Floor Slab Joints (Stairwell)

The assessment of the cross wall / floor slab joint has shown that:
<u>Horizontal Ties:</u> Ineffective *
<u>Vertical Ties:</u> None

The cross wall joint at the location of the stairwell is therefore **<u>insufficient</u>** to pass the assessment for a Consequence Class 3 building.

* The horizontal ties in this area used the same design as for the standard cross wall joints. However, as there is no floor slab on the other side of the cross wall in the stairwell, there is no bar the otherside for the welded bars to connect to. The 'tie' is therefore redundant and ineffective.

Flank Wall / Floor Slab Joints

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

The flank wall joint is therefore **<u>sufficient</u>** to pass the assessment for Consequence Class 3.

Flank Wall (Corner Panel) / Floor Slab Joints

The assessment of the cross wall / floor slab joint has shown that:
<u>Horizontal Ties:</u> None
<u>Vertical Ties:</u> None

The corner flank wall joint is therefore **<u>insufficient</u>** to pass the assessment for a Consequence Class 3 building.

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

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

Horizontal Ties: Effective

Vertical Ties: Not required *

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

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

Cross Wall Vertical Joints

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

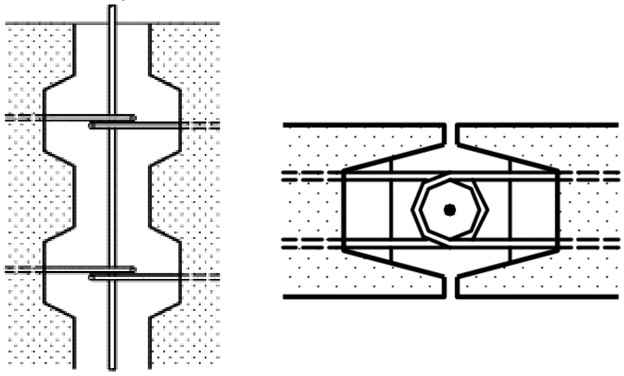


Figure 14 – Typical shear key connection at a vertical joint between wall panels (fib, 2008)

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

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

Spine Wall / Spine Wall Joints

As per Cross Wall.

SHROPSHIRE HOUSE (CONSEQUENCE CLASS 3)			
JOINT TYPE	ADEQUATE HORIZONTAL TIE	ADEQUATE VERTICAL TIE	NOTES
Flank Wall	√	√	Adequate *
Cross Wall	√	X	Inadequate
Wall to Wall Joints	X	X	Inadequate

SHROPSHIRE HOUSE – LPS CRITERION 1 – ADEQUATE PROVISION OF TIES
FAIL

* Flank corner panel found to fail due to no mechanical ties, only main panels pass assessment.

5.3. LPS Criterion 2 – Adequate Collapse Resistance

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

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

The initial assessment was carried out using an over-pressure of 34kN/m^2 to comply with the regulations for accidental loading for building with a piped-gas supply. The calculations show that the structural elements that form Shropshire House are insufficient to resist a loading of this magnitude.

The following table summarises the findings:

	SHROPSHIRE HOUSE	
STRUCTURAL ELEMENT	34KN/M^2 OVERPRESSURE (PIPED-GAS SUPPLY)	NOTES
Floor Slab (Downward)	X	Inadequately Robust
Floor Slab (Uplift)	X	Inadequately Robust
Flank Wall	X	Inadequately Robust
Cross Wall	X	Inadequately Robust

SHROPSHIRE HOUSE – LPS CRITERION 2 – ADEQUATE COLLAPSE RESISTANCE
FAIL

5.4. LPS Criterion 3 – Alternative Load Paths

For a block to pass Criterion 3 the structure must be able to mobilise alternative load paths in the event of an explosion. In the event of an explosion with a piped-gas supply, the bounding enclosure area would be considered to be the entire flat. The overpressure from such an event is considered to act on all elements within this bounding enclosure simultaneously. This could result in three floor slabs and four wall panels being subjected to this loading.

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

SHROPSHIRE HOUSE – LPS CRITERION 3 – ALTERNATIVE LOAD PATHS
FAIL

5.5. Summary of LPS Criteria Checks

Shropshire 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.

SHROPSHIRE HOUSE		
LPS CRITERION	PASS / FAIL	NOTES
LPS 1	X	Inadequately Tied
LPS 2	X	Inadequately Robust
LPS 3	X	Inadequate Mobilisation of Alternative Load Paths

SHROPSHIRE HOUSE – CONCLUSION – FAIL

It can be seen that the block of flats failed LPS Criteria 2 and 3 largely due to the piped-gas supply that is serving the properties, which would generate an onerous overpressure on the structure in the event of an internal gas explosion. A re-assessment of the blocks was therefore carried out to determine the effects of removing the piped gas from the blocks, See Sections 5.6 and 5.7.

5.6. LPS Criterion 2 – Adequate Collapse Resistance – Reassessment for the Removal of the Piped-Gas Supply

Following the failure of the key element checks for the overpressure associated with a piped-gas supply, further calculations were carried out to assess the block of flats for a reduced over-pressure of $17kN/m^2$. This over-pressure would relate to a block without a piped-gas supply. However, as Shropshire House, at the time of inspection, had a piped-gas supply this would have to be removed to comply with the regulations.

The calculations show that the structural elements that form the blocks are insufficient to resist a loading of this magnitude ($17kN/m^2$).

The following table summarises the findings:

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

SHROPSHIRE HOUSE – LPS CRITERION 2 – ADEQUATE COLLAPSE RESISTANCE
FAIL

It can be seen that removal of the piped gas supply allows the flank wall to pass the assessment, as it is adequately robust to resist the reduced over pressure. The cross walls, as they are unreinforced, are reliant on tensile resistance of the concrete and axial compression alone. The cross walls in the upper 8 floors fails the 'key element' checks, although the lower floors were seen to pass due to the increased axial load. The floor slabs remain inadequate.

5.7. LPS Criterion 3 – Alternative Load Paths – Reassessment for the Removal of the Piped-Gas Supply

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

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

SHROPSHIRE HOUSE – LPS CRITERION 3 – ALTERNATIVE LOAD PATHS
FAIL

5.8. Summary of Reassessment

The re-assessment of the blocks has shown that, even with the piped-gas supply stripped from the building, they remain insufficiently robust to resist disproportionate collapse.

It should however be noted that the removal of the gas-supply does reduce the likelihood and magnitude of an internal gas explosion. Whilst not preventing a potential disproportionate event from occurring, it does reduce the risk.

6. NORMAL LOADING ASSESSMENT

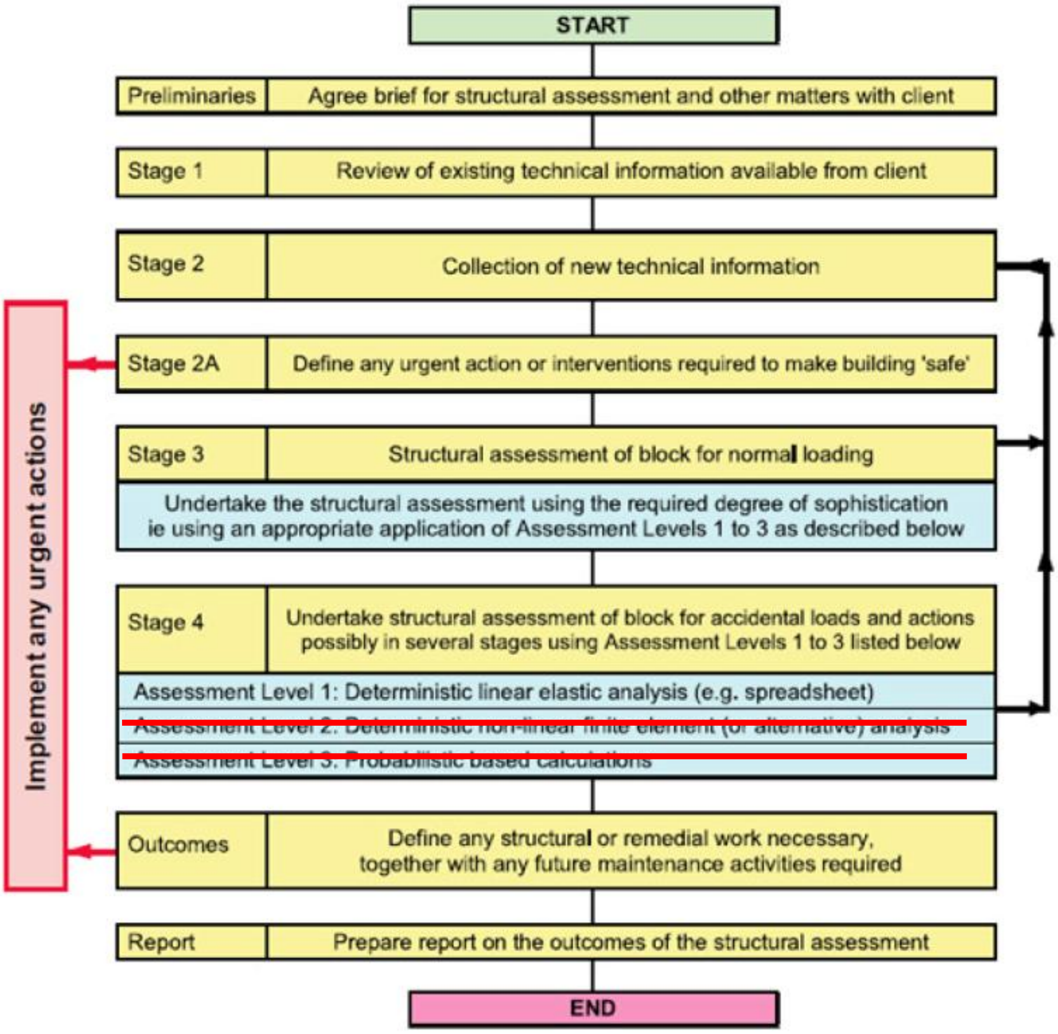
Normal loading is associated to the load from general everyday living situations in a domestic dwelling (1.5kN/m^2 loading on floor slabs, and a conservative 1.0kN/m^2 wind load on flank wall panels). This 'normal loading' does not account for accidental loading events.

Whilst the block has been shown to be insufficiently robust to resist an overpressure associated with an internal gas explosion, the analysis for the block to resist 'normal loading' has been shown to pass the strength checks.

It must be noted that the passing of the 'normal loading' strength assessment does not mean that the blocks are sufficient to pass regulations. The failure of the block for the accidental load assessment means that works must be done to the blocks to bring them up to the required standard. However, as the blocks have passed the 'normal loading' assessment, this may allow the blocks to remain habitable, if required by the client, whilst works are carried out on the blocks.

7. CONCLUSION

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



The outcome of the assessment is that the block in its current state is inadequate to resist disproportionate collapse regardless of whether there is a piped-gas supply serving the blocks or not.

However, it should be noted that the removal of the piped-gas supply would aid to reduce the risk of an internal gas explosion causing a disproportionate collapse event.

The concrete testing has shown that:

- Within the precast concrete panels, the reinforcement is at varying levels of risk of corrosion. This is mainly due to high levels of chloride within the concrete. Minimal concrete cover within the floor slabs is also a contributing factor.
- Within some of the insitu joints, the reinforcement is actively corroding. In some areas the corrosion has been accelerated due to voids in the concrete from items such as rope and polystyrene.
- The precast elements have a characteristic compressive strength of 30.1N/mm^2. [Note this is based on core sampling of the cross walls only. The two flank walls tested were found to have a compressive strength significantly higher – although this will not alter the assessment for the flank wall. The floor slabs could not be cored due to occupied flats above/below.]
- The compressive testing appears to indicate an adequate degree of quality control in the concrete mix during construction, with compressive strengths reasonably consistent.

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

8. RECOMMENDATIONS

8.1. Removal of Piped-Gas Supply

It is our recommendation that the piped-gas supply should be removed from the properties, to be commenced as soon as practicably possible.

However, it should be understood that removing the gas from blocks will be a significant undertaking and cannot be achieved over-night, hence a phased approach to reduce risk is to be recommended.

Best practice for short term measures would comprise:

1. An immediate estate wide ban on the use of any gas cannister/bottles being used or stored within the dwellings, along with a complete ban on any other potentially explosive substances;
2. The removal of gas cookers and replacement with a non-gas source (such as electric, pending confirmation of the adequacy of in-flat electrical circuitry and block distribution systems) – both bottled gas and gas cookers should be viewed as the highest risk as they have the potential to be left on, causing a leak that might then be ignited, causing explosion and excessive pressures being applied on the structures;
3. The installation of interrupter devices linked to gas leak detectors to shut-off valves that will stem the flow of gas to boilers that serve heating and hot water in the event of a gas leak (this viewed as an interim measure until the gas boilers can be removed);

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

4. The renewal of gas boilers with an alternative non-gas heating/hot water source (this may also require investigation of current electrical capacity into the dwellings, blocks and estate);
5. Removal of all gas supplies in the blocks, to a point outside of the curtilage;

8.2. Strengthening Works / Risk-Reduction Measures

In addition to the gas removal works, we advise that the London Borough of Enfield commission a risk analysis, together with a cost-benefit analysis, to be carried out to determine:

a. Whether the risk of disproportionate collapse of the blocks could be acceptably reduced by risk-reduction measures; or
b. Whether strengthening works are required.

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

Strengthening works may take the form of steel strapping of the floors and walls and strengthening of the joints using steel angles.

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

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

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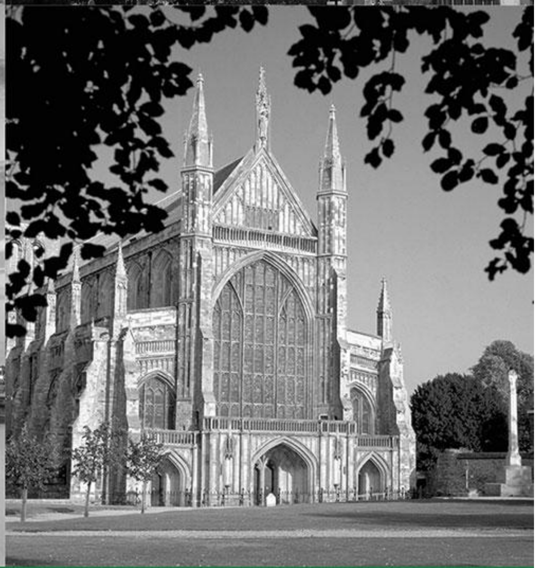

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