RIDGE

BARTON HOUSE

STRUCTURAL ROBUSTNESS ASSESSMENT – PHASE 2

February 2024

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EXECUTIVE SUMMARY

The project brief from Bristol City Council was to enhance the investigative works based on the initial outline reporting carried out to determine the building's ability to resist disproportionate collapse.

The initial reporting had limitations due to the availability of vacant units and the real time available to investigate within the complex live building environment. With the limitations and results of the initial reporting in mind the team has embarked upon a significant regime of further investigations which have set out to qualify the initial reporting.

This has involved investigating six further flats on top of the previous three flats and has therefore provided the team with a greater opportunity to appraise the building in further depth than was available to them previously. This has provided the team with significantly more information than was available beforehand.

Importantly, more locations have been available at critical structural junctions; this has produced a much larger data set for us to appraise the building; crucially these are in areas where we know there might be a weakness.

With the increased information set we have been better able to compare between the historical drawings and the information found on site, and consequently understand key differences between the design intent and the as built structure.

The differences are compared against codes of practice, detailing and written papers that allow an experienced engineers in this field to develop opinions for which calculation is then used to further understand the behaviour of the structure and implement consequent remedial measures.

Our conclusion is that the internal ties are sufficient to satisfy current standards, the flank wall ties given variation in the building construction and material have a marginal shortfall.

With this marginal shortfall the engineering team are using a hierarchical approach to minimise the risk of disproportionate collapse which uses the retrofitted steel frame within the building to assist resisting these loads which will require fire protecting by competent persons.

To summarise, the initial reporting which was limited due to the availability of vacant units. Consequently, locations that could be investigated and the live building environment highlighted issues with the structural ties that could potentially lead to a disproportionate collapse event.

The investigation has highlighted a marginal shortfall in key locations this has led to the use of the steel goal posts to reinforce these areas under the required scenarios to minimise risk of disproportionate collapse increasing the safety of the building.

1. INTRODUCTION

A robustness assessment commissioned by Bristol City Council (BCC), the client, was completed in July 2022 by Ridge & Partners LLP (Ridge) [1] on Barton House. The aim of the assessment was to establish the ability of the structure to resist accidental loads and avoid a disproportionate collapse as per the Building Regulations Approved Document A requirements [2]. The assessment reviewed historic information and carried out a series of intrusive investigations on limited number of vacant flats. The information obtained from the investigations was compared against record information available for the building and a robustness assessment was carried out based on the recommendations of Building Research Establishment (B R E) 511 "Handbook for the structural assessment of large panel system dwelling blocks for accidental loading" [3].

The initial intrusive investigations [1] were unable to consistently identify all the structural ties in the flats made available, as presented in record drawings. Subsequently, the initial report suggested that the building may not have sufficient ties to resist the forces imposed by an accidental event and may experience disproportionate failure. It was advised that a risk analysis is carried out and mitigation measures are put in place to reduce the risk of such failure taking place to acceptable levels.

In 2023 BCC carried out a risk workshop with other consultants, the outcome of which prompted the client to seek further clarification on the existing condition of the building. In November 2023, Ridge was commissioned by the client to carry out follow up detailed intrusive site investigations on available flats at Barton House and confirm the building's structural robustness compliance. The second appointment forms Phase 2 of the intrusive investigations with the initial stage being Phase 1.

As part of this commission, Ridge carried out further intrusive investigations to confirm record drawing information and previous assumptions made. Ridge provided continuous site presence throughout the investigation period and accessed a further 6 No. additional flats in much more detail bringing the total number to 9 flats being assessed between the two phases.

This report provides a historic review of Barton House and lays out the methodology used during the intrusive investigations. A robustness assessment is carried out based on the data provided and further recommendations on any remedial actions that might be required are made.

Limitations

Whilst the investigative works were detailed, with multiple tests carried out in each of the 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.

This report provides an overview of the visual and intrusive surveys carried out and covers only elements of the building which fall within the remit of the structural engineer carrying out the inspection. It excludes fire assessment as this is being carried out by a separate specialist consultant.

The conclusions contained herein are based upon the visual inspection of any visible structural elements of the building. This report does not comment on any non-visible elements or any other structural elements that were not accessible during the site visits.

This report exclusively belongs to Ridge and Partners LLP and is confidential, intended solely for the designated client. Although it can be shared with their professional advisers, its contents must not be disclosed or utilized by any third party without our explicit written consent. Granting such consent does not authorize the third party to rely on the report or confer any rights under the Contracts (Rights of Third Parties) Act. Without such consent, we cannot assume any responsibility towards third parties. Ridge and Partners LLP certify that they have carried out the work contained herein with due skill, care and diligence to their best belief and knowledge based on the time and information available.

2. BACKGROUND INFORMATION ON BARTON HOUSE

2.1. General

Construction at Barton house was completed in June 1958 and at the time, was the tallest building in Bristol. Built by Holland & Hannen and Cubitts Ltd the building comprises of a 15-storey structure with basement, a cast in-situ concrete ground to first storey, whilst the remaining 14 storeys utilise a cross-wall construction method. The orientation of the load bearing walls suggest that very little load is carried by the façade walls where corridors and communal balconies are located.

The block has a T-shaped plan with a total of 98 flats, 84 of which are 2 bed and 14 are 1 bed apartments. The flats are accessed via communal corridors located along the north façade. The lifts and stairs are located at the junction of the T with a second stair located at the base of the "T" with a secondary stair to the east elevation.

Figure 1: Barton House (Google Maps 2022)

2.2. Original Structural Form and Review of Record Information

The first floor consists of a 12" (≈304mm) thick in-situ concrete slab and is conventionally reinforced. The slabs above (levels 2 to roof) are being constructed out of 6" (≈152mm) deep precast concrete hollow beams, described as "Gothic Beams", spanning between cross walls with a clear span of 13 feet (≈3962mm) wall to wall. Each precast beam has 2 bottom plain bars running along the beam's length protruding into the walls either side Figure 2. In order to form the slab, gaps between the beams are filled with in-situ concrete and topped with 25mm of insulation and 50mm thick screed nominally reinforced with hexagonal steel netting of no structural significance.

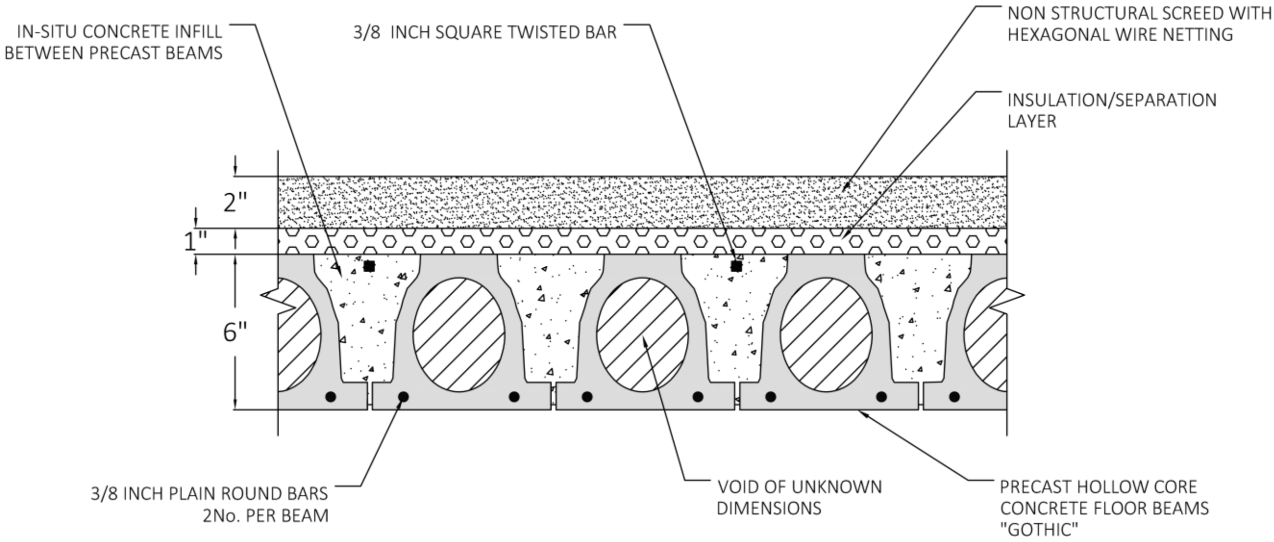

Figure 2: Representation of existing precast concrete slab makeup at tie locations

Load bearing walls, both cast in-situ and precast, are typically 6″ (≈152mm) thick and are reinforced with 3/8″ (≈9.5mm) vertical rebar at 6″ (≈152mm) centres for cast in situ and 9″ (≈228mm) centres for precast panels with ¼″ (≈6mm) horizontal rebar placed at 12″ (≈304mm) centres each face. It appears that the majority of load bearing internal walls were constructed as precast panels with some being cast in situ presumably used to provide lateral stability.

2.3. Historic Defects and Previous Investigations

In May 1968 the entire southeast corner of a 22-storey large panel system (LPS) residential block collapsed due to a gas explosion. Ronan Point was a milestone in the introduction and implementation of disproportionate collapse prevention on all structures. Shortly after the collapse, the UK Ministry of Housing and Local Government released Circulars 62/68 [4] and 71/68, [5] mandating an investigation into the susceptibility to progressive collapse for all existing pre-cast load bearing buildings over 6 storeys.

In 1970 an addendum was issued to CP 116 [6] code of practice for Large Panel precast concrete frames, laying out robustness provisions for such structures. A structural assessment was ordered on all frames taller than 6 storeys which included Barton House. Although there are no records found on the remedial actions taken on Barton house, strengthening works took place in the form of post installed steel frames. This work is believed to have been carried out circa 1970-71. This work is evident throughout the building, and based on the information provided by the council, it appears to be consistent on all floors. During that time, it is believed that all gas was disconnected from Barton House in order to mitigate the risk of an accidental explosion similar to that of Ronan Point.

The design intent of the 1970’s strengthening works is not currently known. However, it is believed that the steel frames are an attempt to provide an alternative load path as described in the circulars. As a result, it is viewed that the steel frames could be utilised to provide an additional line of defence against disproportionate collapse.

In 2020, Ridge was commissioned to carry out a Large Panel System assessment on Barton House. The brief was to carry out an audit on the construction of the block, based on available historic information, followed by detailed intrusive investigations into selected areas. From 2020 through 2022, Ridge carried out periodic intrusive and visual surveys with a report issued later that year [1]. Throughout the duration of the investigations the building remained fully occupied which presented with challenges to the investigation team in terms of availability of vacant flats within which intrusive investigations could be undertaken.

The 2022 report [1] suggested that the structural ties observed could not be deemed effective as some of the horizontal ties were not located in all the flats surveyed (Flat No. 78). Due to the lack of tie provision in some areas it was deemed that horizontal ties were ineffective and thus could not be relied upon to provide sufficient disproportionate collapse protection. In addition, the 2022 report preliminarily disregarded the contribution of the 1970's strengthening steel frame during the assessment with the view that these might be considered during any future proposed strengthening works that might be required on the block.

The report recommended that "a risk analysis, together with a cost-benefit analysis and scheme design for strengthening works, should be carried out to determine whether the risk of disproportionate collapse of the blocks could be acceptably reduced (sic mitigated) by risk-reduction measures; or whether strengthening works are required" [1]. The risk assessment looks to understand if the risk posed is acceptable based on the likelihood of disproportionate collapse, this should include strengthening works. Based on the knowledge of the building during this process the presence of existing strengthening would need to be considered with any shortfalls in information from the first phase. Any additional information that can be identified assists the engineer in forming an opinion as to how the building will generate resistance to disproportionate collapse and / or reduce risk. If risk mitigation measures were not capable of controlling the risk to acceptable levels it was further recommended that demolition of the block was to be considered.

The 2022 report also carried out a durability assessment in the form of carbonation, High Alumina Cement (HAC) content and chloride testing. The report concluded that "the carbonation and chloride content of the concrete elements has shown that the reinforcement is at either a negligible or low risk of corrosion" [1]. In addition, the cement composition was found to be adequate and confirmed the concrete did not contain HAC.

3. INSPECTIONS AND SURVEYS

3.1. Methodology of assessment

The method used to carry out the assessment on Barton House follows the hierarchical approach adopted by B R E 511 [3] as shown in Figure 3 below. The follow up assessment (Phase 2) involved the review of record drawings and any historic information available. A review of historic documentation was also carried out during the initial assessment (Phase 1); this was revisited to ensure that the original design intent was interpreted correctly prior to carrying out any intrusive investigations. Based on this review, critical elements were identified and targeted during the opening up works.

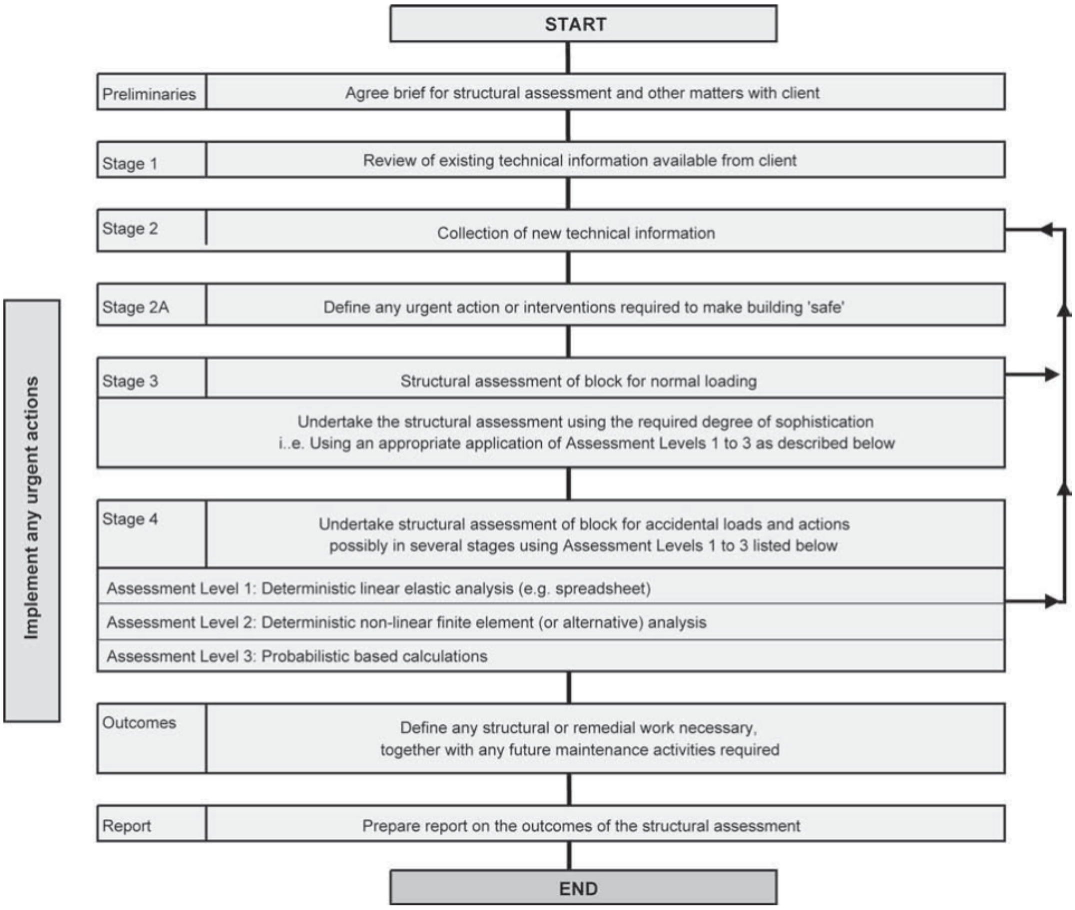

Figure 3: Extract from figure 34 of B R E 511 describing the main steps in the structural assessment process [3]

Several flats were identified in order to carry out the intrusive investigations with the aim to cover most of the critical elements within the structure. Amongst these were the three flats accessed during the 2022 intrusive investigations, flats No. 60, 65 and 78. However, from these three, only flats No. 78 and 60 were made available for access during the second phase of investigations. Flat No. 78 was one of the flats where provision of ties was deemed to be questionable during phase 1. Additionally, flats No. 22, 38, 46, 58, 87 and 91 were accessed as can be seen in Figure 4 below. Combined with Phase 1, approximately 10% of the total number of flats were surveyed during both phases on different floor levels of the structure.

In addition, a measured cover meter survey was carried out on the soffit on a number of flats to establish the existing condition of the cover provided to the precast beam rebar.

The survey was carried out in Flats 22, 46, 58, 78, 87 and 91. The location and results of this survey can be seen in Appendix B.

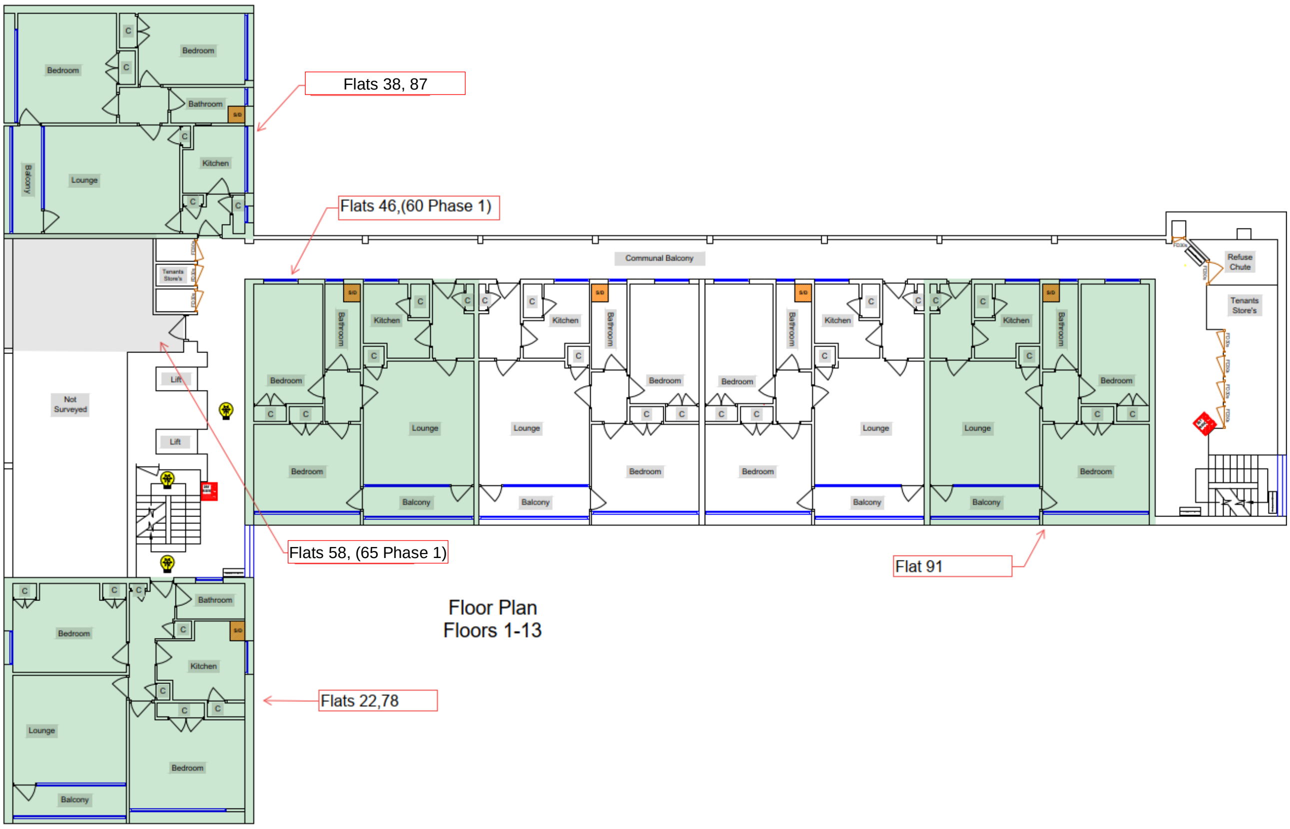

Figure 4: Typical plan view of Barton House, Phase 1 and 2 intrusive investigation locations.

The sequence of works prior to intrusive investigations carried out by the structural team is as follows. Each flat had to undergo an asbestos review to ensure the flat was safe for the intrusive works to be carried out. Typically, asbestos was found in the ceiling coating (Artex coating) and encapsulated underneath the floor screed. Following testing and removal of all asbestos areas an air test was carried out to ensure the flat was safe for access. An electrician was responsible for accessing the flats to ensure that no live wires were in the vicinity of testing. The structural engineering team was responsible for marking the areas of interest to ensure that the critical details are located. A team of operatives was then responsible to break out the screed and remove any finishes on the walls (including masonry). The structural engineering team with the use of a Hilti PS 300 Ferro scan system scanned the areas of interest to identify reinforcement locations and cover levels to the main reinforcement bars. After locating the steel reinforcement, these areas were then exposed by breaking the concrete out to selected areas to confirm the data recorded from the ferro scans.

3.2. Phase 2 Observations

The aim of the phase 2 intrusive investigations was to identify internal and external horizontal ties and compare these against record information. The first three flats accessed were Flats No. 46, 87 and 78. These flats were subjected to intensive intrusive works as they informed the main areas of focus during the investigative works. Of particular interest was Flat No. 78 where during Phase 1 investigations the team did not locate ties at the flank (external) walls. In general, the following observations were documented as below.

Internal (Cross) Wall detail

The internal walls supporting the floor slabs are joined together with a cast in-situ joint as shown in Figure 5. Each beam is being supported by either a 3 ¼ “(≈82mm) steel angle either side or, at party walls, by a masonry wall on one side and a steel angle on the other. Record drawings suggest that these angles are part of the original design intent predating the 1970’s strengthening works. It is likely that the steel and masonry supports were acting as temporary supports to the precast floor beams. Architectural record drawings suggest that the internal masonry walls are 2 ½” (63.5mm) thick with 3/8” (9.53mm) plaster applied on them. However, this was not confirmed via measurement on site. Where measured, masonry walls appear to vary and, in some cases, walls measured up to 3” in thickness.

Typical cover to the internal cross walls was measured to range between 35 – 50 mm inclusive of finishes. Cover measurements in flat No. 60 on internal precast walls measured between 47 – 88mm deep and approximately 40 – 50mm deep vertical joint locations where two precast panels were joined with an in-situ concrete column plug. It is suggested that a minimum cover of 25 mm, excluding finishes, can be expected.

The precast floor beams are stopped short from the internal walls by 1” (25.4mm) with the steel reinforcement of the beams either curtailed upwards or cut short against the wall’s vertical rebar. Typically, in between flats, the walls appear to be cast in situ whilst all the walls within the flat are precast. The former cannot be confirmed through the floor plate as the middle units were not accessed.

Steel ties located at this joint appear to be as described in the record information. These ties consist of straight 3/8” square twisted high yield bars. The ties extend, typically, 800mm either side of the walls and are located in all flats surveyed. They are spaced, typically, at 13 ½” (≈343mm) centres and are located on every other trough of the precast floor units. A horizontal bar, located at the top of the floor slab, is placed just underneath the internal ties within the cast in-situ joint. This bar is believed to serve as a placement bar and has no other structural benefit.

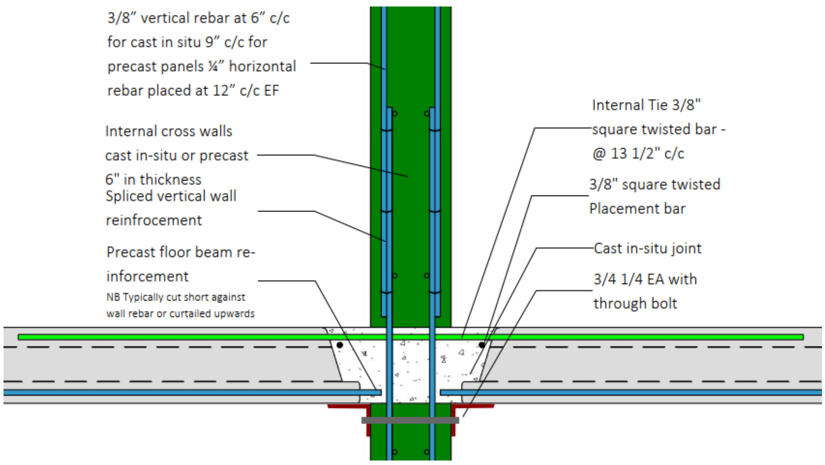

Figure 5: Observed Internal Wall to Floor Detail of Barton House

External (Flank) Wall detail

Similar, to the internal wall joints, flank wall to floor joints are also cast in-situ joints as shown in Figure 6. In this location, floor beams are being supported by a masonry wall with approximately 40mm air void between the two. Where measured, this wall appeared to be 3" (76mm) thick. Architectural record drawing information reviewed for a typical floor plan on the west wing, appears to show a masonry thickness of 3" based on a Flat type No. 78. This seems to correspond well with the measurement taken on site. As with the internal walls, the precast floor beams are stopped short by 1" (25.4mm) with the steel reinforcement of the beams either curtailed upwards or cut short against the wall's vertical rebar.

Typical cover to the external walls was measured to range between 35 – 40 mm. It is suggested that a minimum cover of 30 mm, excluding finishes, can be expected.

Steel ties located at this joint appear to be as described in the record information, however, their location within the element is not as per the original design intent as they are typically placed closer to the inside face of the walls. The ties consist of L-shaped 3/8" square twisted high yield bars. The ties extend, typically, 800mm beyond the wall towards the slab. Opening up works in the ceiling were carried out to determine the length of the horizontal bars into the walls below. Although the ends of each bar were not exposed to their full length, it was observed that each bar is anchored into the lower walls by at least 250mm below the underside of the floor slab.

Typically, horizontal bars are spaced, at 13 ½" (≈343mm) centres and are located on every other trough of the precast floor units. A horizontal bar, located at the top of the floor slab, is placed just underneath the internal ties within the cast in-situ joint. This bar is believed to serve as a placement bar and has no other structural benefit.

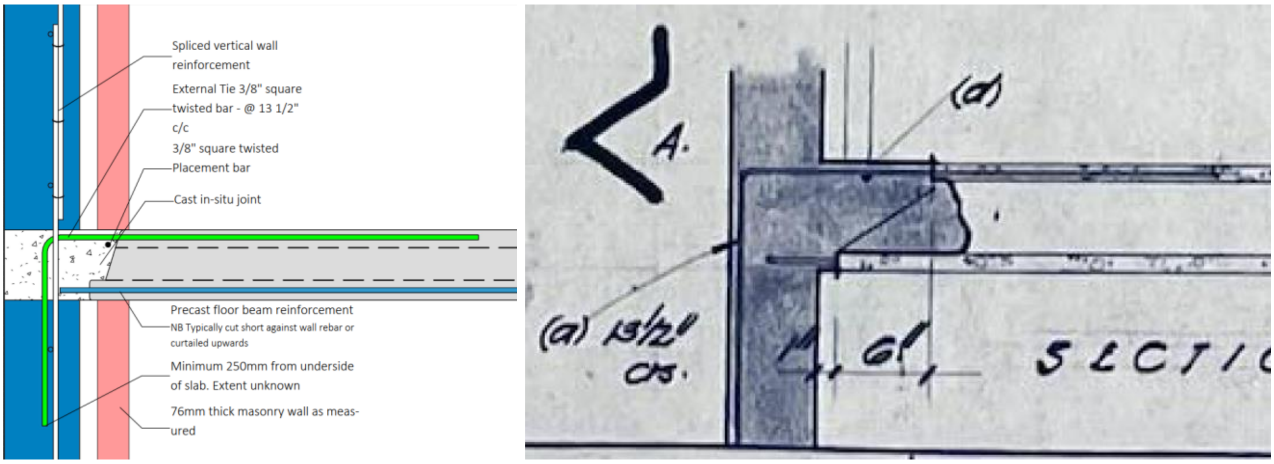

Figure 6:Observed External Wall to Floor Detail as Observed (left) as Intended, extract from record drawings (right)

Interim (Internal) Flank Walls

Towards the western end of the block, the precast concrete floors change span direction and are being supported by internal cross walls running perpendicular to the previous spans. A comparison with record drawings suggests that the precast beams at Flat No. 78 were placed transversally to what was originally intended. This change does not necessarily suggest an error and could signify a later change in the design given the later roof layout alters.

During phase 1 of the investigations, ties were generally observed throughout on flat No. 65 [1]. However, no record is included in the report on the exact location of the ties and whether the junction between Flats No. 65 and 64 (the latter being equivalent of No. 78) was investigated. Intrusive investigations during phase 2 were carried out in Flat No. 58, directly below flat No. 65. The investigation did locate ties on the wall boundary between flats No. 57 and 58; however, further investigation on the soffit of that location failed to locate the ties coming from the flat above, No. 65. There was an expectation by the design team that as this wall is an end panel, L-shape ties should have been provided. This would have been apparent at the junction between the floor slab and the wall as was discovered elsewhere. There could be multiple reasons why the ties in the soffit of flat No. 58 were not located. The difficulty of locating the ties at this instance does not suggest that these are not present but merely that the investigation was inconclusive.

Flank Walls Adjacent to Stores

Each floor level of Barton House contains two communal store areas, one in each stair core. These areas were not accessed during any of the intrusive investigation phases. Towards the west, the stores are located adjacent to an internal flank wall forming the boundary of flats similar to 58 and 65. Towards the east end of the block, the communal stores are abutting an external flank wall.

Each storeroom is separated from each other by masonry walls spanning from floor to ceiling and from the communal corridors by a 30 min fire rated door. All communal corridors are exposed to the atmosphere as they are not enclosed.

Areas such as stores might be considered to pose a risk against storing items that might generate an accidental action on the structure. However, internal compartmentalisation of the stores is likely to limit the exposure of the flank walls to any adverse loading arising from such events. In addition, compartmentalisation within the stores is likely to limit the exposure of the flank wall panels to each store footprint. As a result, exposure of the flank walls at these locations is not considered to be significant. It is suggested that an accidental event in this area is likely to be localised and may not extend to the entire wall panel. However, it is recommended that a ban on storing gas cylinders is enforced throughout the building to mitigate against the risk of such accidental events. In addition, it is suggested that each store door is vented to avoid the accumulation of any dangerous gases.

Other observations

During intrusive investigations in Flat 91, a horizontal crack was discovered in one end of the floor slabs Figure 7. The crack is located approximately 300mm away from the face of the support and is formed relatively parallel to the wall. A visual check was carried out on the flat below to establish whether the crack has penetrated the floor slab in its full depth or is restricted to the top layer. From the visual inspection carried out on the flat below it appears that the crack is limited to the top layer of the slab and has not penetrated through. It is unknown when this crack was formed and whether it is a historic defect or a result of any intrusive investigations carried out on the floor above. Although the structural team did not inspect the area until the screed was removed, areas where the screed was not removed in the vicinity of this defect suggested that the screed might also have been affected. If the crack is indeed historic it is likely that it was formed due to localised hogging moments experienced close to the support rather than due to excessive shear forces. This is because, shear failure is typically brittle and does not provide warning. It is recommended that this defect is repaired by installing additional reinforcement between every other trough that is not currently occupied by existing horizontal ties.

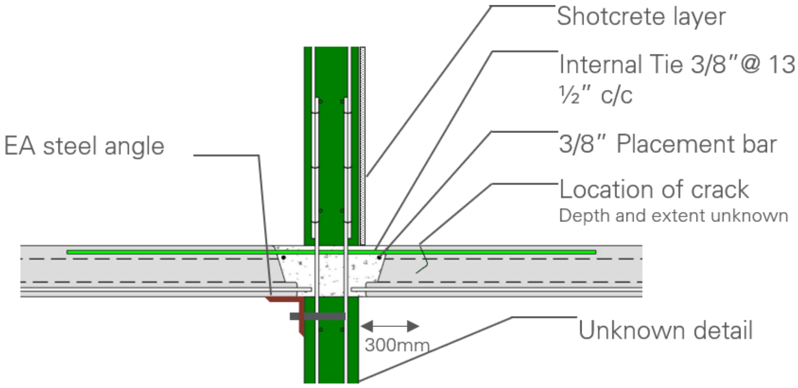

Figure 7: Observed Internal Wall Detail at Flat No. 91

Opening up works on some walls suggests that initially cover levels might not have been satisfactory. In some areas cover depths were measured as low as 5mm; however, in all observed affected areas a layer of 40mm thick shotcrete was installed in front of the wall providing sufficient cover for durability purposes. This type of defect was observed in flats No. 91 and 78. The time of the installation of this additional layer is not known as it could be part of quality control during construction or installed during the 1970s strengthening works.

Removing the concrete and exposing reinforcement from the majority of the cast in situ walls was proven to be an arduous task. This suggests that the strength of concrete in these areas is higher than that of a typical precast wall element.

Slab Soffit Cover Metre Survey

A cover meter survey was carried out upon the request of the client in order to establish typical cover to the soffit of the slab reinforcement. Typically, the concrete cover within the precast beams measured circa ½" (12.7mm). The measurements presented in Appendix B appear to be in excess of that measurement. This is due to the fact that different types of finishes have been applied to the soffit within the different apartments. Where finishes were removed, the cover to the reinforcement provided appeared to be consistent with the as build requirements of ½" in thickness.

4. DISRPOPORTIONATE COLLAPSE ASSESSMENT

4.1. Structural assessment requirements

B R E 511 [3] clearly identifies the requirements against which each structure should be assessed for. Although Barton House is a non LPS structure, disproportionate collapse requirements apply to all buildings. Satisfying one of the three criteria then the structure is assessed to satisfy requirement A3 of building regulations Approved document A. The three criteria set up by B R E 511 are as follows:

LPS Criterion 1: There is adequate provision of horizontal and vertical ties to comply with the current requirements for Class 2b buildings 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.

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.

As the piped gas has been removed from the structure, one of the main risks of disproportionate collapse has been mitigated. The structure is required to be assessed against the above three criterion including a reduced overpressure of 17 kPa.

As the block contains a basement it should be kept well-ventilated to avoid gas buildup and therefore mitigate against an overpressure exposure of 34 kPa. This also includes any other areas that gas might be able to accumulate.

4.2. Structure Classification

Based on BCA technical guidance note 21 "The building regulations 2010 – England & Wales requirement A3 – Disproportionate collapse" [2] Barton House is classified consequence class 2b (15 Storey over basement) as shown in Figure 8. Approved document A stipulates that in addition to the Class 2a requirements for horizontal ties, Class 2b structures require effective vertical ties.

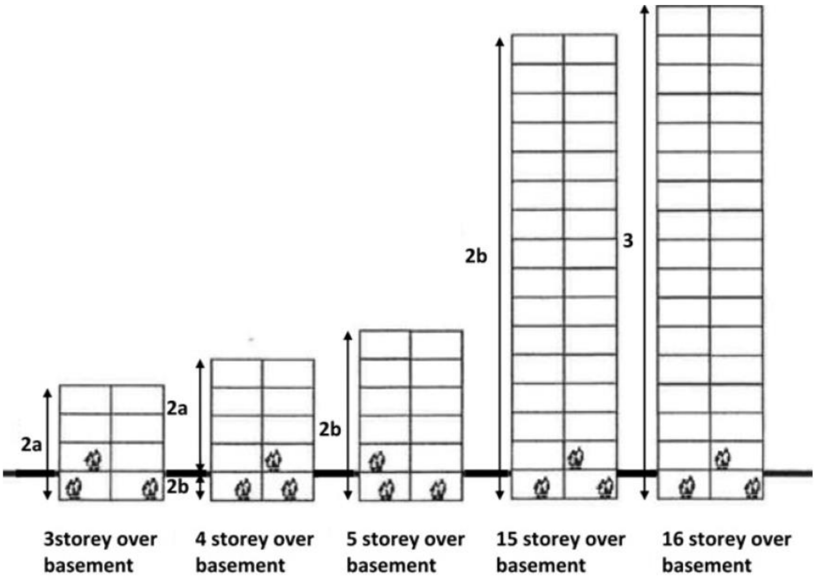

Figure 8: Disproportionate Collapse Classes of robustness measures [7]

4.3. Materials

Information provided by record information suggests that the cube compressive strength of the concrete used at 7 days is 5000 lib/in^2 which is approximately 34.5 N/mm^2. Typically, concrete strength of CEM I type concrete at 7 days is 2/3 of target strength. Thus, target strength is estimated at 51.7 N/mm^2 suggesting that the concrete type used at the time was either a C40/50 or a C35/45. The Phase 1 intrusive investigations included testing a series of concrete cores. Based on core data for a 1:1 core ratio and by information and following the requirements of BS 6089:2010 [8] it is estimated that external walls have a compressive cylinder strength of a 39.4 N/mm^2. Converting this value to a 2:1 core suggests a compressive cylinder strength of 32.31 N/mm^2 based on the standard deviation, target strength values and therefore considered appropriate.

The strength of the steel reinforcement was not tested. Based on information provided on the concrete centre's website [9] for cold worked deformed bars of less than 3/8" in thickness, it suggests a yield value of 70000 psi (482.6MPa) whereas plain round mild steel bars would have a yield value of 36000 psi (248.2Mpa). By adopting a material factor as described in BS EN 1993-1 of γ_m = 1.15, a yield strength of 482.6/1.15 = 419.9 N/mm^2 is adopted for cold worked deformed bars and 248.2/1.15 = 215.8 N/mm^2 for plain bars.

4.4. Design Assessment Scenarios

There are four different wall configuration scenarios applicable to Barton House based on its geometry and construction type. These are highlighted in the drawing attached in Appendix C and are also summarised in the list below:

- Scenario 1: External flank walls panels. These are located at the junction between the floor slab and external walls where the wall forms part of the slab's supporting system. They are critical lateral and vertical load bearing elements running from ground to roof floor levels.
- Scenario 2: Internal flank wall panels. These are internal wall panels located where the floor slab changes span direction. Although these walls are internal, they support only one end of the floor slab with the other end spanning parallel to the

wall. For all intents and purposes these walls are treated in a similar manner to that of external walls.

- Scenario 3: Flank Walls adjacent to stores. Similar to scenarios 2 or 3 with the difference being that the walls are located adjacent to communal stores.
- Scenario 4: Internal Cross walls. These form part of the load bearing system of the building and provide floor slab support at both ends of the wall. These panels can be either cast in situ or precast wall panels and can be interrupted (i.e. transferred) at first floor level.

4.5. Basis of design assessment LPS Criterion 1

Although the majority of ties have been provided as intended, a workmanship error was consistently observed when exposing the flank wall horizontal ties (Scenario 1). The original design intent was for the horizontal ties to be lapped at the rear face of the wall (Figure 6 left). Doing so would have ensured that the force applied in the ties would be equal throughout the tie itself. However, the as built condition suggests that the horizontal ties were installed just at the back of the front face wall reinforcement (Figure 6 right) providing an embedment of approximately 60 mm. The location of the flank wall ties in relation to the front of the wall exacerbates the force experienced by the ties. The method to analyse such condition and its effects is described below.

Analysis Methodology

There is little information in the codes about how to check the capacity of the concrete and the behaviour of the node of an L-shaped embedded bar. Following a review of design literature, the most appropriate way found to analyse such condition was presented in a 2008 paper by Gary J. Klein titled "Curved Bar Nodes" [10], the concept of which is shown in Figure 9 below. The concept behind the curved bar nodes is based on a strut and tie method. When two connecting elements are of equal geometry the force in the tie is uniform and equals to the force applied at the face of the support. When one of the two elements are of different geometry, the force between T_1 and T_2 is relevant to the lever arm corelation between C_1 and C_2 respectively as the Product $T_1 * d_s = T_2 * d_w$ and hence $T_2 = T_1 * d_s / d_w$. Since ds is approximately 2.16 times greater than d_w the force exerted in the tie T_2 should be 2.16 times of T_1. A more codified approach to this concept can be found in PD 6687-1:2010 figure B [11].

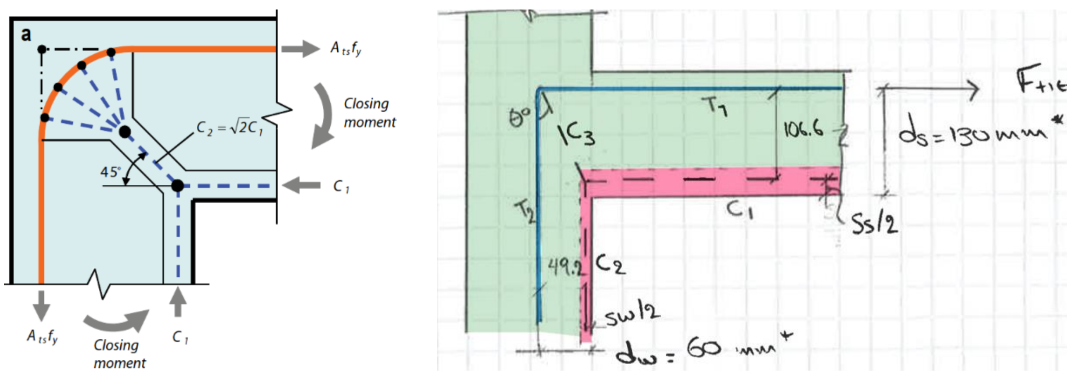

Figure 9: Strut-and-tie model of forces concept (left) [10], flank wall strut-and-tie analysis model of Barton House (right)

4.6. Adequacy Provision of Ties

The effectiveness of both horizontal and vertical ties is assessed against the Eurocode document BS EN 1991-1-7:2006 [12] and BS EN 1992-1-1:2004 [13] and in accordance with PD 6687 [11].

A review of the vertical ties was carried out during the initial phase of intrusive investigations and was found to be adequate. The same document rendered the effectiveness of horizontal ties ineffective due to the difficulty of locating them in Flat No. 78. Following the Phase 2 intrusive investigations it was established that horizontal ties are present in Flat No. 78 as described in the record drawings. Moreover, presence of the ties was established and confirmed in other flats as well. A minor exception to this was at the party wall ceiling of flat No. 58 with flat No. 57. Although ties were located on the floor, this was not possible at ceiling level. A review of phase 1 report suggests that flat No. 65 (situated above Flat No 58) did have horizontal ties, although it is unclear in the report whether the floor at that junction was part of the investigation.

Horizontal Ties

BS EN 1992-1:2004 clause 9.10.2.4 [13] requires that "*Edge columns and walls should be tied horizontally to the structure at each floor and roof level. The ties should be capable of resisting a tensile force $f_{tie,fac}$ per metre of the façade*". This suggests that horizontal ties should resist 60 kN per metre equating to a $f_{tie,fac}$ = 20.57 kN. The maximum applied force in the internal ties is equal to 20.57 kN (Scenario 4). However, the force within the ties located at flank walls (Scenarios 1, 2 and 3) is increased by 2.16 times $f_{tie,fac}$ = 44.57 kN due to the possible as built condition. The estimated capacity of each on tie is equal to 43.78 kN if the material factor is not considered and 38.1kN if the material factor is applied. This suggests that the internal ties are sufficient (Scenario 4) whilst the flank wall ties (Scenarios 1, 2 and 3) are 87% (γ_m = 1.15) to 98.2% (γ_m =1.0) effective or 115 - 101% overutilized respectively.

Based on ACI 318 [14] for CCT (Compression, Compression, Tension) strut and tie nodes, the concrete strength of the external tie node suggests that the maximum applied stress in the concrete can be calculated as $0.85*\beta_n*f_{ck}$. The use of the ACI code in this instance is proposed as it provides a codified approach to dealing with strut and tie models as well as being the most conservative code between Eurocodes and the American standards. Hence the maximum concrete stress is estimated at 16.49 N/mm^2 whereas the applied stress is in the region of 17.2 N/mm^2. This suggests that the overall effectiveness of the joint is approximately 96% or 104% utilised.

Vertical Ties

The requirement for vertical ties contained within the guidance documents is reproduced herein and is as follows. a) "*Each column and each wall carrying vertical load should be tied continuously from the lowest to the highest level. The tie should be capable of carrying a tensile force equal to the design load likely to be received by the column or wall from any one storey under accidental design situation [i.e. loading calculated using BS EN 1990:2002+A1:2005, Expression (6.11b)]*" [15]. The following expression applies:

$$E_d = G_k + A_d + \psi_{1,1}Q_{k,1} + \Sigma\psi_{2,i}Q_{k,i}$$

where

G_k = characteristic value of a permanent action

A_d = design value of accidental action

ψ_1 = frequent value of a variable action

Q_k = characteristic value of a variable action

ψ_2 = quasi-permanent value of a variable action

Based on the analysis the maximum load applied in the vertical ties is in the range of 46.8 kN per metre run. The maximum capacity per metre for two layers of 3/8" plain bars is equal to 134.53 kN suggesting that the vertical ties are 35% utilised.

4.7. Basis of design assessment LPS Criterion 2

Although it was felt that with a more rigorous analysis, LPS criterion 1 could be achieved throughout the building, an examination of LPS criterion 2 was also carried out to establish the resilience of the building against disproportionate collapse for Scenarios 1 and 2. The assessment considers the contribution of the 1970's steel strengthening works and checks against an accidental over pressure load of 17 Kpa.

Ridge has not surveyed the entirety of the block to establish whether steel frames exist throughout the building. As such, Ridge does not have visual evidence of the condition of each frame as this would require full access to all the flats within the block, the logistics of which has proven to be challenging. However, historically the client, BCC, has carried out spatial surveys and has produced drawings suggesting the presence of steel frames throughout the building. As a result, it was felt that steel frames could be relied upon to provide structural support during an accidental load case as described by BS EN 1990:2002+A1:2005, Expression (6.11b). The presence of steel frames within the communal stores is currently not known.

Analysis Methodology

A simple analysis was carried out to establish the loads imposed on the slabs, walls and steel frames during an accidental event. The following two load cases were considered during the steel frame wall and slab assessments.

Combination 1 (Downforce): 1.0*Dead Load + 1.0*Accidental Load + 0.5*1.5*Live load

Combination 2 (Uplift): 1.0*(-Dead Load) + 1.0*Accidental Load + 0.0*1.5*Live load

Where, dead load was calculated to be 4.0 kPa, live load 1.5 kPa and Accidental load 17.0 kPa

The methods used to establish the capacity of the slab is based on the codified approach. A steel yield stress of 248.2 N/mm^2 for plain bars and a concrete strength of 32 N/mm^2 was adopted. For the uplift combination the floor slab was checked against the provisions of section 12 of BS EN 1992-1-1:2004 for plain and lightly reinforced concrete structures. The shear capacity of the slab was checked against clause 12.6.3 expression (12.5) with the axial prestress load set to zero.

The steel frames are typically installed at midspan between the internal and external walls, reducing the effective span of the slabs by half. As such, the effective span was taken as 2.0 metres between the steel frames and external/internal walls. No moment continuity was taken into consideration at the steel frame supports as there is no provision of top reinforcement. A small steel bar, identified during the investigations at the top of each precast beam, is believed to act for lifting purposes and as such its contribution was ignored.

The analysis suggests that in a downwards motion, the slabs are capable of withstanding an accidental overpressure of 17.0 kPa and are 80% utilised. During the same event, the uplift forces acting on the ceiling would be resisted by the tensile resistance of the concrete itself. The latter was found to be utilised 100% which is a marginal pass.

With regards to the 1970's steelwork, the following assumptions apply. The steel beam size is assumed to be 152x30 UC S275 spanning marginally under 4.0 metres. The beam is checked against the overpressure of 17kPa whilst ignoring imposed and dead loads. This is because the beams were installed post completion of the block, and any dead load would have already materialised and carried by the concrete floor. Based on those assumptions the imposed moment on the beam is circa 66.9 kNm with an elastic capacity of 68 kNm suggesting that the beams are 98% utilised. By considering any contribution from the live loads during an accidental event it suggests that the beam will be fully utilised.

Finally, the internal and external walls were checked against a 17 kPa overpressure. During the assessment any contribution of the vertical loads was ignored and the capacity of the precast and cast in situ walls was estimated based on record information. Assuming the walls are spanning top to bottom as simply supported, a moment capacity of 15.77 kNm and 23.48 kNm is achieved in the precast and cast in-situ panels respectively. An overpressure moment of 13.8 kNm is applied during an accidental event making both walls satisfactory in resisting accidental loads.

As the external wall ties were found to be marginally ineffective, it is suggested that the contribution of the steel frames at flank wall locations, for Scenarios 1 and 2, is utilised to ensure that the building performs satisfactorily against disproportionate collapse throughout. In doing so it is paramount that the steel frames adjacent to any flank walls, shown in Appendix C, Figure 10, are fire treated to ensure that they perform adequately in the event of a fire.

4.8. Sensitivity Analysis

It is noted that the utilisation of horizontal ties is marginally overstressed. However, this assessment is based on deterministic linear analysis based on a level 1 assessment. It is suggested that if a non-linear analysis was carried out the results would be less conservative, and that the capacity of the joint might be sufficient to withstand the forces applied.

In addition, it is noted that the forces applied to the joint is based on a codified minimum requirement. The true forces acting on the joint due to an accidental event might be smaller than that of the minimum requirement.

Finally, it can be seen from the simplified analysis approach presented in section 4.5 that the joint capacity is reliant on the as installed position of the horizontal tie and is very sensitive to its effective depth. It is estimated that for the joint to be satisfactory a minimum effective depth of 62mm is required. The current chosen value for the effective depth was taken arbitrarily as the minimum dimension observed during the site investigations. This does not mean that all the ties are installed at this minimum value, and it is likely that the joint has adequate capacity to resist the codified accidental loads.

5. CONCLUSION

Ridge and Partners were commissioned to carry out a follow up detailed intrusive site investigation on available flats at Barton House and confirm the building's structural robustness compliance. For this purpose, a review of historic documentation and previous reporting was carried out alongside intrusive investigations on a total of 9 flats. The assessment was carried out based on B R E 511 recommendations and a review of LPS criteria was carried out based on the findings of the investigations.

Barton House is classed as a Consequence Class 2b structure, as described in Building regulations Approved document A. This classification requires for effective vertical and horizontal ties to be provided. An assessment on both horizontal and vertical ties carried out herein, as required by B R E 511 against LPS criterion 1, concluded the following remarks:

- The horizontal ties provided between internal (cross) walls to slab joints was found to be effective against disproportionate collapse.
- The horizontal ties provided between external (flank) walls to slab joints was found to be marginally ineffective against disproportionate collapse. Further non-linear calculations, if carried out, might prove that these joints are marginally effective. Furthermore, sensitivity analysis suggests that conservative assumptions made during the robustness assessment might affect the overall results.
- Vertical ties provided between wall panels were found to be effective against disproportionate collapse.

The position of the horizontal ties has a significant effect on the forces experienced by the steel ties. The assumption made in this assessment is that all the as built ties were installed with a relatively small cover. However, during the phase 2 site investigations, only one tie was found to have such small cover. A nominal increase in the cover of the tie would make the external ties effective against disproportionate collapse.

With the marginal shortfall in the effectiveness of the ties provided in the flank walls a hierarchical approach was employed by utilising the 1970's strengthening works to assist resisting the building collapse condition at these locations. Slab, wall and steel frame elements were checked against B R E 511 LPS criterion 2. The walls performed adequately whereas the slabs were found to be at capacity (100% utilised). The steel frame, based on conservative assumptions, was also found to be fully utilised.

As these elements are being used to perform a structural function it is recommended that all steel frames adjacent to flank walls, internal or external (Scenarios 1 and 2) are fire treated in accordance with the fire specialists' recommendations and that any fire protection is installed by qualified and competent persons. An exception to this is the internal flank wall, adjacent to both core stores, where the exposure to an accidental load case is not considered to be significant and as such no intervention is currently recommended.

Following a careful consideration of historic interventions carried out in the building (gas removal and steel strengthening) in combination with the findings of this report it is suggested that the building meets the requirements of LPS criteria 1 for the internal ties. At flank wall locations (Figure 10) and by taking into account the contribution of 1970's strengthening works, it is suggested that the building meets the requirements of LPS criteria 2. As such, it is suggested that the building is at a relatively low risk of disproportionate collapse and therefore Barton House would behave adequately during an accidental event.

Four different wall configuration scenarios were assessed against the LPS criteria 1 to 3 as described in B R E 511. The description of each scenario and their assessment against the criteria has been summarised in Table 1 below.

Table 1: Conclusion of Disproportionate Collapse Assessment against B R E 511

SCENARIOS	LPS CRITERION	ASSESSMENT	COMMENTS
Scenario 1 External flank walls panels	**Criterion 2** Adequate Strength to Resist Accidental Loads	**Sufficient**	External Flank walls are marginally ineffective against LPS Criterion 1. By taking into account the contribution of 1970's strengthening works, it is suggested that the building meets the requirements of LPS criterion 2
Scenario 2 Internal flank wall panels	**Criterion 2** Adequate Strength to Resist Accidental Loads	**Sufficient**	Internal Flank walls are treated in a similar way to the external walls. It is proposed that the contribution of 1970's strengthening works is considered to meet the requirements of LPS criterion 2
Scenario 3 Flank Walls adjacent to stores	**Criterion 2** Adequate Strength to Resist Accidental Loads	**Sufficient**	Accidental exposure to this wall panel is not considered to be significant and as such disproportionate collapse is unlikely. It is proposed that store doors are vented to avoid accumulation of dangerous gases and that all mobile gas cylinder storage is banned.
Scenario 4 Internal Cross walls	**Criterion 1** Adequate Ties within Joints	**Sufficient**	The existing frame has effective vertical and horizontal ties to meet LPS Criterion 1 at these locations

6. RECOMMENDATIONS

The following recommendations are being proposed for Barton House.

Immediate actions:

- It is recommended that historic strengthening works at flank wall locations are utilised to protect against the effects of disproportionate collapse due to accidental loads for Barton House. Doing so will require for steel frames to be adequately protected against corrosion and fire. These frames have been annotated in Appendix C, Figure 10.
- It is recommended that strict measures are adopted in banning the use of any liquid gas devices (such as mobile heaters) that might be currently in operation at Barton House and that these are monitored on a regular basis.

- It is recommended that adequate basement ventilation is provided to reduce the risk of accumulation of any flammable gases in this area.
- It is recommended that all areas used for communal storage are vented externally to reduce the risk of accumulation of any flammable gas.
- It is recommended that all residents are informed by an awareness campaign which may include information on restricted items within flats and storage areas etc.

Medium term actions:

- It is recommended that a risk analysis is carried out for Barton House to measure and monitor all elements that have the potential to cause an accidental event and that suitable preventative measures are taken to mitigate against these. The risk assessment will include but not be limited to the information provided by the robustness structural assessment, as described herein, and fire assessment carried out by specialist fire engineers.

Long term actions:

- It is recommended that a risk analysis is kept up to date throughout the remainder of the lifespan of the structure.

This report does not assess the structure against fire as this is being carried out by a specialist fire consultant. It is recommended that this report is read in conjunction with the fire consultant’s report and both findings are adopted in full.

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APPENDIX A

PHOTOGRAPHS OF INTRUSIVE INSPECTIONS

Photograph 1: Internal Wall Junction Horizontal Tie

Photograph 2: External Wall Junction Horizontal Tie

Photograph 3: External Wall Junction Curtailment of Tie

Photograph 4: Typical Temporary Support Angles

Photograph 5: Typical Layer of Shotcrete in front of Low Cover Panel

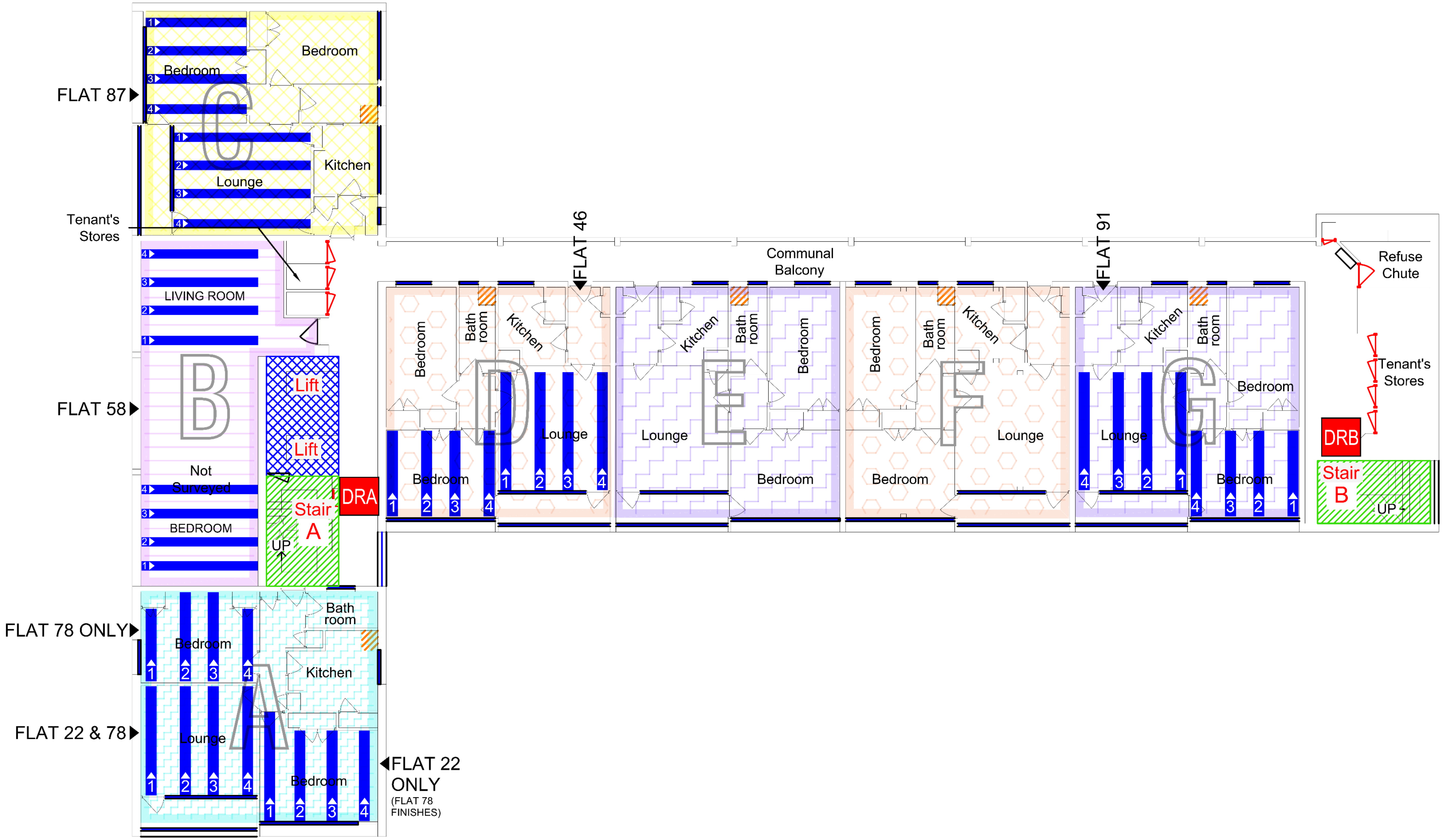
Photograph 6: Flat No. 91 Floor Crack

Photograph 7: External (Flank) Wall Cast In-situ Panel with Masonry Wall

Photograph 8: Precast Floor Beams Reinforcement Provision

APPENDIX B

SLAB SOFFIT COVER METRE SURVEYS



Key Plan 1: Overview of cover metre scan locations

Table 2: Cover metre scan measurements part 1/2

Note: Beam pairs do not align between scans - beam pair 1 represents the first beam pair encountered in the scan
Note: Where a value is affixed thus '*' the reading was distorted and may not accurately depict the cover depth

		Beam Pair 1		Beam Pair 2		Beam Pair 3		Beam Pair 4		Beam Pair 5		Beam Pair 6		Beam Pair 7		Beam Pair 8		Beam Pair 9		Beam Pair 10		Beam Pair 11		Beam Pair 12		Beam Pair 13	
Flat 22	Bed Scan 1	30	38	35	37	34	34	30	32	29	29	34	34	34	34	35	35	32	35	37	33	42	36	34	37	39	41
	Bed Scan 2	35	34	25	27	26	27	34	28	31	31	34	36	35	24	29	28	27	30	34	31	27	34	30	39	30	35
	Bed Scan 3	-	36	38	36	26	29	31	30	34	23	27	28	31*	31*	38	33	36	27	29	31	35	30	30	26	25	29
	Bed Scan 4	37	37	32	29	26	25	32	35	28	28	30	32	31	30	35	25	29	32	37	30	30	29	27	34	28	28
	Bed Scan 5	-	33	26	31	29	29	34	32	38	28	22	30	29	29	30	29	30	32	28	31	33	35	36	34	31	33
	Living Room Scan 1	28	29	29	28	32	36	39	28	24	30	32	32	30	31	30	34	29	31	33	32	35	31	30	29	21	34
	Living Room Scan 2	28	33	34	33	31	30	27	29	31	33	30	31	32	35	35	32	33	36	32	37	36	33	38	40	34	29
	Living Room Scan 3	30	34	37	41	39	32	34	34	36	38	35	34	33	40	33	33	32	37	32	38	38	42	50	44	34	23
	Living Room Scan 4	-	40	48	48	43	39	39	35	45	46	39	38	34	39	32	33	35	35	34	38	41	40	42	43	34	27
Flat 46	Bed Scan 1	-	35	31	30	29	30	32	28	26	31	26	29	27	32	33	31	39	36	26	25	26	40	34	40	40	27
	Bed Scan 2	-	36	32	32	32	32	33	34	33	38	33	32	32	32	37	40	44	35	33	29	35	39	33	33	37	33
	Bed Scan 3	-	37	31	30	32	32	26	33	32	22*	33	32	27	28	34	38	33	27	26	30	23	34	37	27	35	23
	Bed Scan 4	28	31	31	32	26	31	28	25	28	30	25	26	29	29	27	30	29	30	27	29	32	30	21	26	25	28
	Living Room Scan 1	27*	36	35	32	15*	36	29	32	28	30	27	25	26	32	32	30	33	29	26	27	31	38	34	35	28	26
	Living Room Scan 2	-	31	33	32	28	27	26	29	27	33	30	29	22	30	29	28	30	35	37	19*	31	34	37	33	33	33
	Living Room Scan 3	-	27	30	36	28	31	25	29	28	27	34	32	32	35	27	38	32	28	29	28	27	30	30	32	30	35
	Living Room Scan 4	29	25	27	35	28	26	26	29	31	28	31	31	35	31	25	26	32	29	29	28	28	28	29	29	27	30
Flat 58	Bed Scan 1	-	33	32	32	29	30	29	30	34	20*	34	29	34	33	32	26	28	27	20	41	27	31	33	28	30	31
	Bed Scan 2	30	29	28	26	28	26	33	22	30	30	32	31	32	26	26	25	24	35	24*	33	34	31	31	28	31	30
	Bed Scan 3	27	28	27	25	26	25	28	25	25	24	24	25	26	26	24	24	24	29	32	32	30	26	29	29	23	27
	Bed Scan 4	32	36	32	29	28	27	31	25	24	23	29	29	30	34	30	34	17*	Err*	32	34	27	29	31	29	26	32
	Living Room Scan 1	29	37	34	30	31	31	36	40	31	21	33	20*	30	30	31	30	32	34	28	30	31	34	31	33	38	34
	Living Room Scan 2	-	36	30	30	32	31	30	35	27	27	33	27	32	34	33	33	27*	27*	30	26	29	32	33	31	40	40
	Living Room Scan 3	28	37	33	32	27	30	29	33	31	31	33	28	27	34	32	36	35	35	35	33	34	34	25*	36	36	38
	Living Room Scan 4	-	31	26	26	29	25	28	33	29	31	36	26	26	30	24	29	29	31	25	26	30	30	26	30	34	37
Flat 78	Bed 1 Scan 1	-	29	27	29	28	28	29	30	24	32	30	29	35	30	23	24	22	22	23	31	31	25	21	25	32	24
	Bed 1 Scan 2	-	31	30	27	32	29	30	30	32	32	32	26	29	33	28	31	35	33	35	33	35	35	32	31	35	30
	Bed 1 Scan 3	-	28	27	21	30	25	27	27	31	33	32	27	30	30	26*	26*	26	26	24	30	30	27	32	28	24	40*
	Bed 1 Scan 4	-	14*	19	27	28	29	31	29	30	24	24*	24*	30	25	28	31	31	30	23	27	28	30	25	27	29	27
	Bed 2 Scan 1	27	38	31	30	14	31	33	31	34	34	27	23	28	25	24	30	28	34	32	32	40	33	29	30	29	27
	Bed 2 Scan 2	32	39	36	33	18	34	37	34	36	36	29	30	30	30	25	33	30	36	36	30	35	32	32	28	36	34
	Bed 2 Scan 3	-	34	32	33	29	30	33	38	36	29	34	37	32	32	25	27	32	30	30	30	29	26	21	20	29	34
	Bed 2 Scan 4	-	35	34	28	29	34	31	35	25	33	32	34	34	33	28	31	31	33	31	39	31	35	35	29	28	35
		Note: Living Room Scans aborted due to fireboarding finishes installed																									
Flat 87	Bed Scan 1	-	29	29	29	30	18	23	29	27	28	21	25	22	24	21	26	24	32	31	25	25	20	28	22	27	28
	Bed Scan 2	-	31	32	32	28	27	27	28	28	22	24	28	27	27	31	28	25	25	33	27	25	26	22	30	22	27
	Bed Scan 3	-	24	21	22	25	28	25	29	30	26	23*	28*	26	30	31	26	32	30	29	26	30	22	22	22	25	30
	Bed Scan 4	-	22*	23	21	19	28	28	31	31	30	26	25	24	28	27	26	28	31	29	27	-	28	25	24	27	27
	Living Room Scan 1	30	27	42	19*	30	28	31	25	29	30	32	32	28	32	31	32	28	30	31	30	27	27	29	30	28	28
	Living Room Scan 2	-	26	28	36	32	31	32	32	27	26	27	28	25	28	31	30	31	34	11	37*	33	38	28	27	27	28
	Living Room Scan 3	-	24	25	29	27	28	28	26	28	27	27	25	24	29	28	30	34*	18*	33	23	30	27	26	19	22	24
	Living Room Scan 4	28	32	30	26	25	27	34	27	27	28	29	23	22	26	27	31	29	28	32	33	28	28	32	24	27	28
Flat 91	Bed Scan 1	-	27	27	27	28	27	26	28	27	24	30	23	21	25	26	28	34	28	29	27	27	23	31	26	24	25
	Bed Scan 2	-	31	25	28	28	29	28	25	25	27	30	27	27	34	41	30	32	32	32	24	34	27	31	30	31	25
	Bed Scan 3	26	24	24	25	26	25	24	29	22*	22*	22	31	30	28	32	24	27	25	26	27	29	19	25	24	29	22
	Bed Scan 4	27	22	22	27	28	21	21	32	28	22	22	28	26	20	25	28	27	25	24	27	29	26	29	20	25	24
	Living Room Scan 1	30	29	31	34	20*	38*	33	31	30	33	29	30	26	30	29	29	30	28	30	25	30	30	25	25	29	29
	Living Room Scan 2	27	31	30	28	29	31	30	29	34	24	24	34	32	27	22	28	25	31	30	30	37	21	59*	31	28	30
	Living Room Scan 3	26	29	24	27	25	31	31	30	34	30	19*	19*	43	26	23	35	33	28	24	27	24	24	30	28	28	26
	Living Room Scan 4	22	24	28	26	27	26	29	21	25	35	30	22	27	31	31	29	23	25	26	24	30	22	24	24	28	30

Table 3: Cover metre scan measurements part 2/2

Note: Beam pairs do not align between scans - beam pair 1 represents the first beam pair encountered in the scan
Note: Where a value is affixed thus '*' the reading was distorted and may not accurately depict the cover depth

		Beam Pair 14		Beam Pair 15		Beam Pair 16		Beam Pair 17		Beam Pair 18		Beam Pair 19		Beam Pair 20		Beam Pair 21		Beam Pair 22		Beam Pair 23		Beam Pair 24		Beam Pair 25		Beam Pair 26	
Flat 22	Bed Scan 1	-	35	30	37	31	21	22	End																		
	Bed Scan 2	28	35	30	26	27	End																				
	Bed Scan 3	25	28	29	36	24	23	32	End																		
	Bed Scan 4	34	39	29	29	35	End																				
	Bed Scan 5	30	34	34	33	34	30	27	32	29	34	27	33	36	49	36	End										
	Living Room Scan 1	32	30	30	31	25	28	29	32	28	32	33	48	35	End												
	Living Room Scan 2	29	37	33	32	31	39	32	28	30	31	29	39	36	30	31	End										
	Living Room Scan 3	32	27	30	29	32	34	27	26	29	42	38	31	35	37												
	Living Room Scan 4	27	33	32	32	40	39	36	37	36	38	38	40	41	41	39	End										
Flat 46	Bed Scan 1	27	26	27	29	31	End																				
	Bed Scan 2	32	26	27	31	38	End																				
	Bed Scan 3	22	30	32	25	29	End																				
	Bed Scan 4	30	29	End																							
	Living Room Scan 1	32	33	29	22	31	26	29	29	21	32	34	33	25	28	30	31	34	33	End							
	Living Room Scan 2	26	31	25	28	34	26	31	26	30	33	31	37	35	25	28	28	30	31	33	34	29	26	End			
	Living Room Scan 3	30	33	31	43	18*	18*	30	39	28	30	25	29	31	29	25	25	26	35	32	34	33	29	End			
	Living Room Scan 4	28	27	29	30	31	31	29	30	28	27	26	27	32	32	25	27	34	26	28	27	32	End				
Flat 58	Bed Scan 1	37	23	30	27	27	35	32	32	26	26	23	32	29	27	End											
	Bed Scan 2	33	37	37	30	24	27	27	28	27	25	End															
	Bed Scan 3	30	31	30	27	20	28	29	24	31	19	End															
	Bed Scan 4	32	30	28	25	24	30	27	28	29	23	End															
	Living Room Scan 1	33	26	28	35	28	23	26	33	End																	
	Living Room Scan 2	39	Err*	36	31	28	32	25	31	26	27	30	26	35	End												
	Living Room Scan 3	39	31	33	34	31	28	30	33	29	32	31	35	34	End												
	Living Room Scan 4	36	31	30	35	37	33*	42	37	37	33	35	42	37	End												
Flat 78	Bed 1 Scan 1	27	25	26	28	26	23	20	27	End																	
	Bed 1 Scan 2	27	28	32	33	30	31	28*	31	End																	
	Bed 1 Scan 3	22*	34	30	27	28	37	End																			
	Bed 1 Scan 4	31	27	27	27	25	26	24	29	28	28	25	End														
	Bed 2 Scan 1	29	28	31	36	27	26	29	36	15	End																
	Bed 2 Scan 2	35	42	34	29	18*	18*	40	End																		
	Bed 2 Scan 3	34	33	34	30	25	32	37	30	28	End																
	Bed 2 Scan 4	30	30	30																							
		Note: Living Room Scans aborted due to fireboarding finishes installed																									
Flat 87	Bed Scan 1	33	23	29	20	23	25	End																			
	Bed Scan 2	29	31	31*	34*	35	26	25	End																		
	Bed Scan 3	25	25	31	25	28	29	32	29	End																	
	Bed Scan 4	29	28	31	26	31	29	31	33	32	24	End															
	Living Room Scan 1	30	34	30	30	31	30	30	25	29	27	33	34	29	27	25	30	28	25	20	22	29	33	End			
	Living Room Scan 2	27	28	31	31	27	28	29	30	27	24	25	28	29	26	29	30	27	25	23	29	23	30	End			
	Living Room Scan 3	24	27	29	27	32	32	28	27	28	22	24	24	27	27	28	27	29	30	27	25	24	28	25	27	End	
	Living Room Scan 4	28	24	30	26	27	23	27*	30*	26	22	26	27	26	28	29	30	34	24*	36	28	End					
Flat 91	Bed Scan 1	25	29	27	30	31	End																				
	Bed Scan 2	22	30	30	32	30	End																				
	Bed Scan 3	19	27	29	26	End																					
	Bed Scan 4	20	21	23	27	End																					
	Living Room Scan 1	30	25	29	25	31	28	27	29	21	28	25	28	29	26	23	25	27	28	21	32	End					
	Living Room Scan 2	31	30	30	29	28	28	34	26	28	31	28	30	29	30	31	25	29	25	26	25	24	27	End			
	Living Room Scan 3	29	31	31	24	28	31	29	28	25	33	32	29	29	30	29	30	27	26	25	25	24	27	End			
	Living Room Scan 4	26	27	29	34	28	29	30	32	31	29	28	30	25	25	31	30	29	30	End							

APPENDIX C

CRITICAL STEEL FRAME LOCATIONS

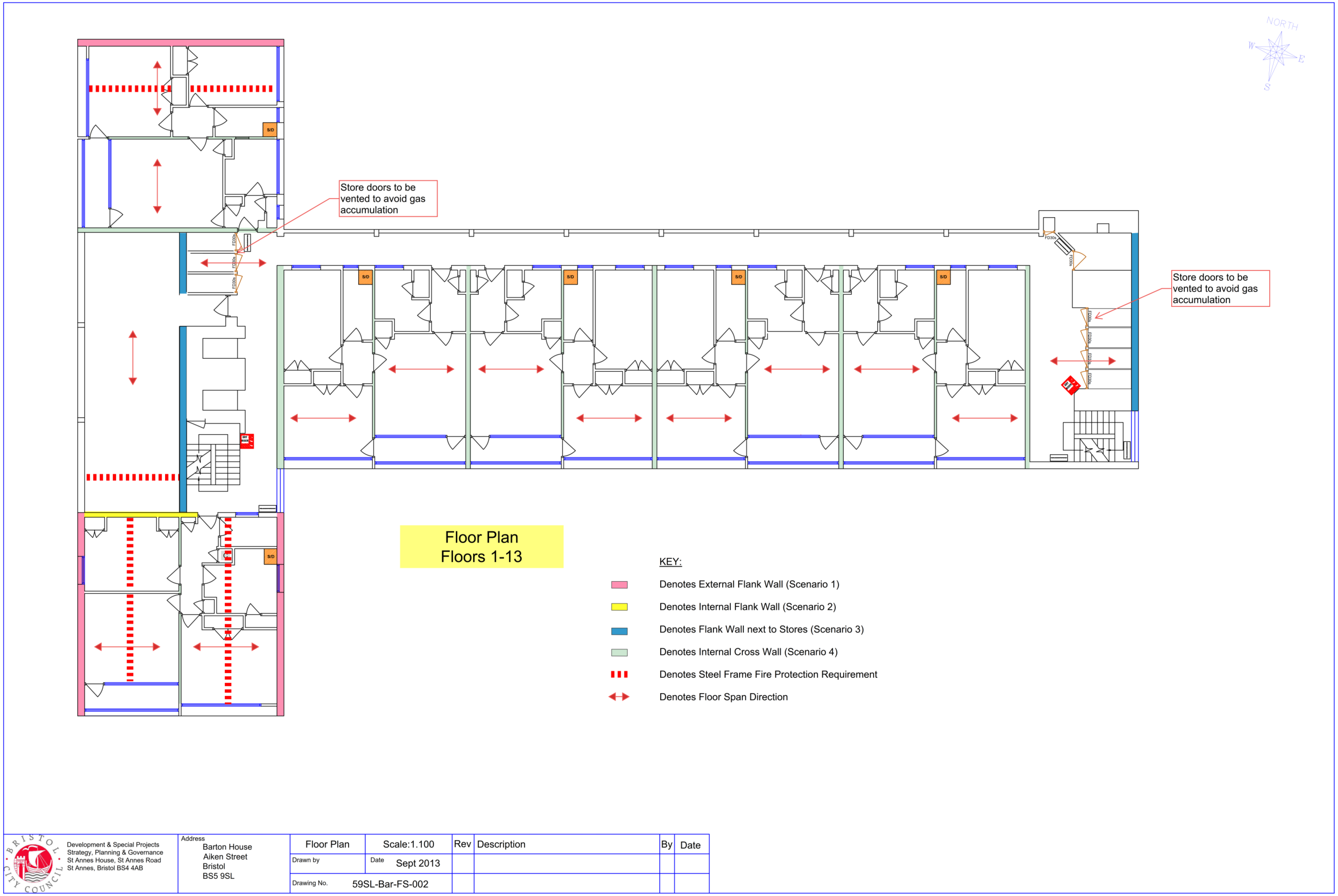


Figure 10: Plan of steel frame fire encasement requirements

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