RIDGE

PROPERTY & CONSTRUCTION CONSULTANTS

Homes for Haringey

BROADWATER FARM ESTATE
STRUCTURAL ROBUSTNESS ASSESSMENT
KENLEY AND NORTHOLT REPORT

April 2018

Prepared for

Homes for Haringey
108 Gloucester Road
London
N17 6GZ

Prepared by

Ridge and Partners LLP
Partnership House
Moorside Road
Winchester
S023 7RX

[redacted]

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1. INTRODUCTION

Ridge and Partners LLP (Ridge) was appointed by Homes for Haringey (HfH) to carry out structural investigations to determine the robustness of twelve dwelling blocks on the Broadwater Farm Estate, Haringey, London. The appointment came after the publication of a similar report in August 2017 for four LPS towers in the Ledbury Estate within another London Borough; this initial study showed that the Ledbury structure failed to satisfy the three-criteria set out in Building Regulations Approved Document A for disproportionate collapse. Owners of similar LPS dwelling blocks, including Homes for Haringey (HfH), were therefore, advised to seek professional advice regarding the safety of their assets.

This report specifically addresses the two tall towers, Kenley and Northolt.

The Broadwater Farm Estate is comprised of two tall high-rise blocks (each eighteen storeys above an insitu concrete podium):

- Northolt
- Kenley

and ten medium/high-rise blocks (all between four and six storeys above an insitu concrete podium):

- Croydon
- Lympne
- Debden
- Hornchurch
- Hawkinge
- Manston
- Martlesham
- Rochford
- Stapleford
- Tangmere

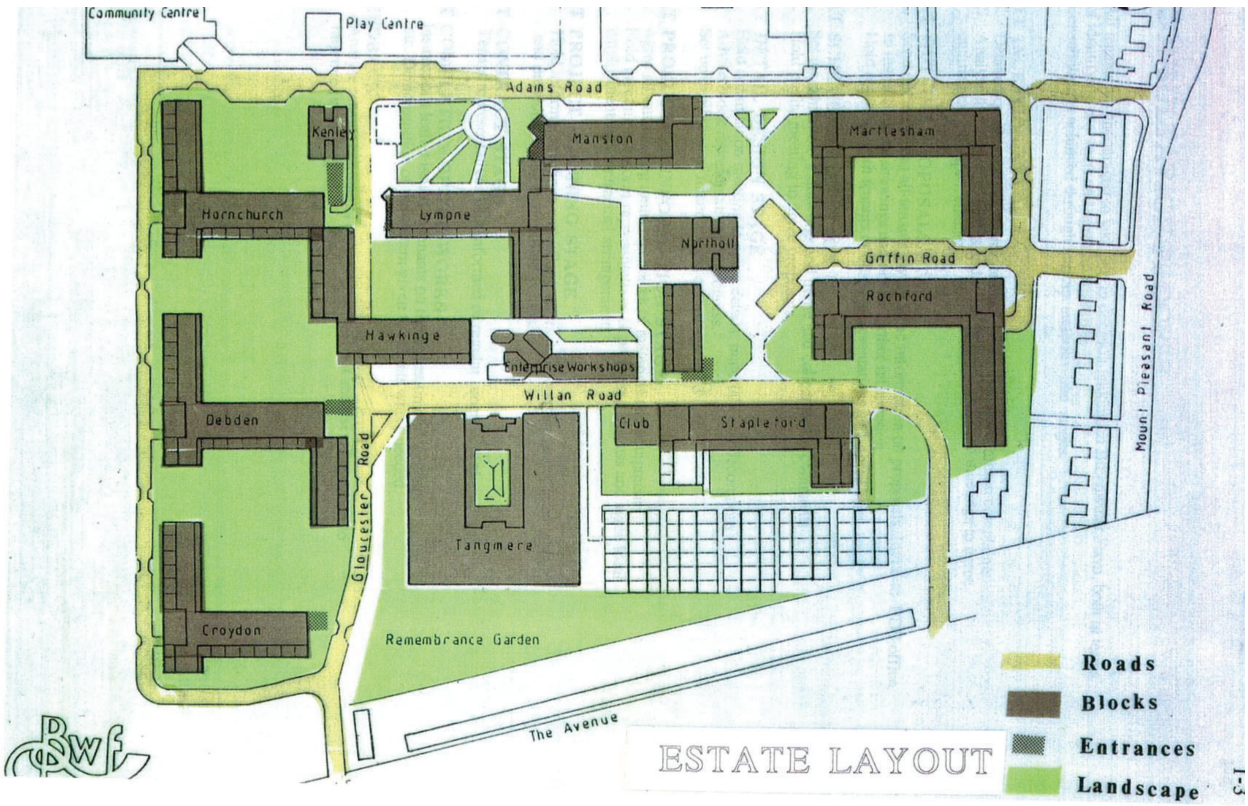

Figure 1 – Broadwater Farm Estate Layout (Haringey Council Building Design Service, 1985)

It is understood that the twelve Large Panel System (LPS) dwelling blocks were built by the Contractor, Taylor Woodrow-Anglian (TWA), for Haringey London Borough Council between 1968-1972. Northolt and Kenley have been confirmed as being constructed using the Larsen-Nielsen system. All ten of the lower rise blocks have a piped-gas supply to at least a portion of the structure (reported as having been in place since their construction). The two tall high-rise blocks do not have a piped-gas supply.

The Larsen-Nielsen system was also used by Taylor Woodrow-Anglian to construct Ronan Point, a 23-storey block of flats in Newham, London, which suffered an internal gas explosion in 1968. The explosion caused progressive and disproportionate collapse, killing four and injuring a further seventeen residents. Following the partial collapse there were a series of changes made to structural design codes and regulatory standards. Several documents have also been published by the BRE, which examine the causes of collapse and provide guidance for the assessment of structures under accidental loading. These documents have been reviewed and used to assess the LPS blocks during this investigation and are referenced as an appendix.

An archive box of information received from HfH was also reviewed for the assessment. Although much of the contents were noted as architectural, some information relating to structural elements were discovered. However, full construction drawings and calculations for the blocks at Broadwater Farm Estate appear not to have been kept by Haringey London Borough Council. [redacted]

In the absence of the construction details of the blocks Ridge proposed to subject each block of flats on the Broadwater Farm Estate to both invasive and non-invasive investigation works. Desktop assessments, including calculations, were then carried out to determine the robustness of the structures based on the findings.

Throughout the duration of the exploratory investigations the blocks, as a whole, remained inhabited by residents. This presented challenges to the investigation team in terms of availability of vacant flats within which intrusive investigations were undertaken.

A key criterion for the assessment of the blocks was the 'over-pressure' loading that is applied to the structure in the case of explosion. If blocks have a piped-gas supply, even to a portion of the structure, then the building is subject to an over-pressure test of 34kN/m^2. Buildings without a piped-gas supply, or a basement, are subject to a lower over-pressure of 17kN/m^2.

2. EXECUTIVE SUMMARY AND CONCLUSION

The twelve dwelling blocks located on the Broadwater Farm Estate have been assessed for their robustness to resist accidental loading from over-pressure, such as an internal gas explosion, and their susceptibility to progressive collapse. The blocks were constructed from a Large Panel System (LPS) by the contractor, Taylor Woodrow-Anglian (TWA), for Haringey London Borough Council between 1968-1972.

In 1967 Ronan Point, an LPS block of similar construction to the blocks at Broadwater Farm Estate, suffered progressive collapse caused by an explosion of gas in one of the flats. This event sparked a series of changes to legislation related to the design of new LPS structures and the assessment of the existing LPS building stock. These documents have been reviewed and used in the assessment contained within this report.

In the absence of the construction details of the blocks Ridge subjected each block of flats on Broadwater Farm Estate to both invasive and non-invasive investigation works to determine its construction. The findings of these on-site investigations were then used in the desktop study to justify the robustness of the block.

The table below summarises the findings of our investigations in the two 18-storey blocks and notes recommendations for further works:

BLOCK NAME	34KN/M^2 OVER-PRESSURE	17KN/M^2 OVER-PRESSURE	NOTES
KENLEY	N/A	✓	No piped gas within block.
NORTHOLT	N/A	X	No piped gas within block. Strengthening works required.

X=Failed Test, ✓=Passed Test, TBA=To be announced, N/A= Not applicable (no gas)

The blocks have 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 blocks need 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.
    - Consequence Class 3 – Kenley and Northolt

- 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. (Matthews & Reeves, 2012)

In the context of new build high rise design, the building regulations requires a systematic risk assessment to be undertaken for class 3 structures such as Kenley and Northolt. Whilst this is not a retrospective requirement it is good practice to consider risks to disproportionate collapse, such as vehicular impact damage which could then be mitigated using vehicular barriers for example. A draft risk assessment has been provided under separate cover.

The calculations for the 'key element' checks have been carried out using British Standards that are more akin to the design codes that the structure would have been originally designed to rather than the Eurocodes.

The 18-storey blocks were grouped together for reporting but were assessed separately based on the finding from the intrusive and non-intrusive investigations. The results of the analysis are as follows:

Kenley

- LPS Criterion 1

  The joints between the loadbearing elements in Kenley were found to possess an effective horizontal tie. However, the vertical tie in the cross-wall joints was found to be insufficient, and in the case of the downstand beam the connection was missing entirely. Therefore, the connections in Kenley do not pass LPS Criterion 1.

- LPS Criterion 2

  The 'key elements' checks carried out on the loadbearing members in Kenley show that the elements are sufficiently robust to resist an over-pressure of 17kN/m^2 associated with a block with no piped-gas supply with the exception of the downstand beam. The block therefore generally satisfies LPS Criterion 2.

- LPS Criterion 3

  Kenley has been shown to satisfy LPS Criterion 2. An assessment for Kenley against LPS Criterion 3 was, therefore, not required as the blocks need only satisfy one of the three LPS Criteria.

Following the assessment of Kenley the block generally passes LPS criterion 2 with the exception of the downstand beam, which will require remedial strengthening works for the block to fully pass this criteria.

Northolt

- LPS Criterion 1

  The intrusive investigation into the joints between loadbearing elements showed that the insitu concrete zone contained no reinforcement linking the panels together. The connection between the elements is therefore solely reliant on the remedial strengthening angles bolted to the elements at the joints.

  However, the investigation showed that the strengthening angles were not installed at the base of the cross-wall panels, and the bolting of the strengthening angle to the flank wall was insufficient. In addition, the reinforcement that was retrospectively detailed to form a horizontal tie within the floor slabs was found not to be installed in all slabs that were inspected.

  Therefore, the connections within Northolt are insufficient to satisfy LPS Criterion 1.

- LPS Criterion 2

  The cross-walls and flank walls within the block have been shown to be unreinforced. The floor slabs are unreinforced in the top, and the bottom reinforcement has been shown to resist ‘normal’ loading only. All elements in the block are, therefore, insufficiently robust to resist the 17kN/m^2 over-pressure associated with an internal gas explosion in a block with no piped-gas supply.

- LPS Criterion 3

  The assessment undertaken for LPS Criterion 2 shows that the cross-walls, flank walls and floor slabs fail under the 17kN/m^2 over-pressure. If an explosion occurred in the living room of the corner units a flank wall and a cross-wall (as well as the floor slabs) would fail. In this instance no alternative loadpaths could be mobilised, potentially resulting in a disproportionate collapse event for the block as all support in removed or compromised.

  From the appraisal carried out in accordance with the BRE report 511 Northolt does not pass the LPS criteria, therefore we would recommend that a study is undertaken that looks at strengthening options for the block in order to pass a minimum over-pressure of 17kN/m^2.

3. 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.

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

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

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

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

BRE Report 107: Part 2 produced in 1987 also provided non-mandatory guidance on the assessment of LPS blocks. This included methodology for inspection of the joints between elements and procedures to evaluate the findings. This report also confirmed the requirement to assess all LPS blocks over four storeys, bringing this in line with Section D17 of the Building Regulations.

The latest requirements for disproportionate collapse are defined in Building Regulations Approved Document A – Structure. This divides building usage types into consequence classes, with differing levels of assessment required for disproportionate collapse. The consequence class table can be seen in Section 8.1.

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

4. THE BLOCKS ON BROADWATER FARM ESTATE

4.1 Brief History of Broadwater Farm Estate

In the 1960s Haringey London Borough Council commissioned the construction of two tall high rise, and ten lower dwelling blocks on the Broadwater Farm Estate. The twelve dwelling blocks were constructed by Taylor Woodrow-Anglian (TWA) between 1967-1972 using the Large Panel System (LPS), specifically thought to be the Larsen Nielsen system (The University of Edinburgh, 2017).

The design of the blocks saw that no habitable rooms were located on the ground floor due to the high water table in the area. The ground floor of each block was instead used as a carpark under the ten medium/high-rise blocks and a service area for the two high-rise blocks. The ground floor was constructed of an insitu concrete podium, upon which the precast units of the LPS were built up from. Another original design feature of the Broadwater Farm Estate was a series of walkways, which connected the blocks at first floor level. The walkways were subsequently demolished in the late 80s and early 90s as part of the regeneration of the estate.

In the years since construction, various remedial works and regeneration schemes have been carried out to the blocks at Broadwater Farm. These include bolting of the external non-loadbearing walls back to the slabs to reduce the movement of the walls caused by thermal expansion by way of steel angles resin fixed to the wall panel and underside of floor units, and over-cladding of the blocks with a rendered insulation system. We are also advised that firestopping works have also been carried out throughout the estate in the 1980s.

In 2006 the management of Broadwater Farm Estate was transferred to Homes for Haringey (HfH) when it was established as an Arms Length Management Organisation (ALMO) to manage all of Haringey's council housing.

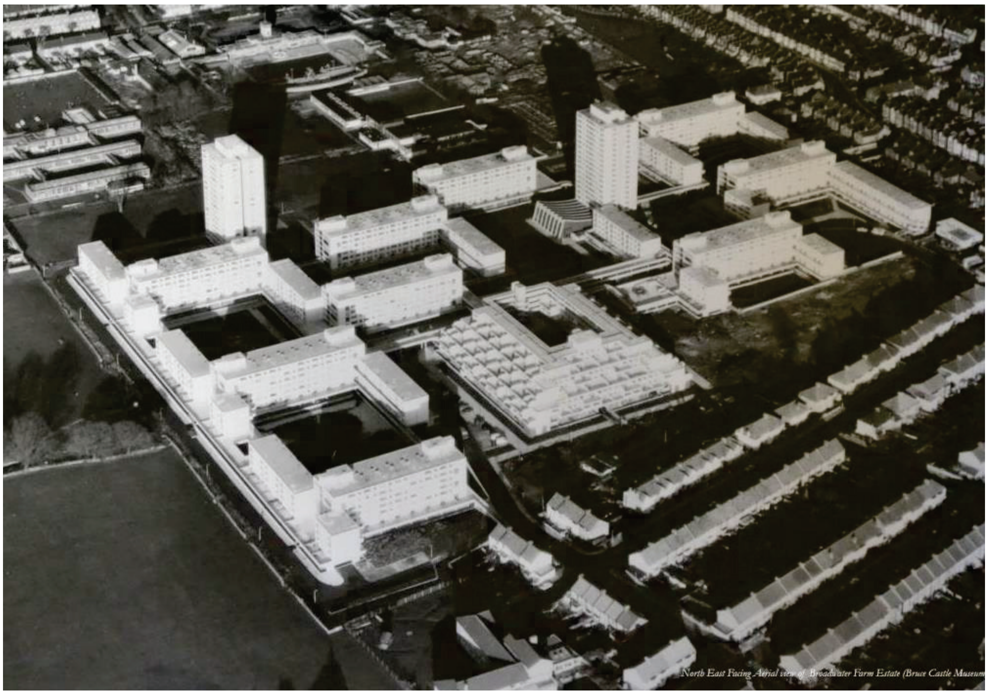

Figure 2 – Historic Aerial Image of Broadwater Farm Estate in 1975 (Coll, 2014)

4.2 Review of Historic Information

An initial desktop study was carried out using an archive box of information received from HfH. Much of the information was noted as architectural, however some structural elements were noted as outlined below:

- The structural repairs that detail bolting to be carried out to the non-structural cladding panels of the blocks between 1986 and 1987 was noted as having been provided to restrict excessive movement.
- No mention is made of reinforcing the panels to resist explosion forces from the design codes of practice.
- The edge detail observed within the information shows the external wall being supported by the wall under or flying past the slab edge. No support or restraint is offered by the hollow core structural slab to the non-structural cladding panel, other than that by the restraint cleat detailed.
- Reports outline the dry packing to structural joints as being adequate.
- Carbonation is noted as being less than 5mm at the time of survey.
- Chlorides were noted as less than 0.4%.
- Recommendations for repair to areas with minimal cover were made.
- A structural investigation into the two tall high-rise blocks, Kenley and Northolt, was carried out in 1985 to determine their robustness.

A section of the 1985 report summarising the findings from the structural investigation into Kenley states that "In regard to explosion resistance, calculations based on the working drawings confirm that the building was designed to resist a minimum equivalent static pressure of 5lbs/sq.in (34kN/sq.m) and thus complies with current building regulations. The appraisal calculations carried out covered in particular the flank walls, the middle and end bays of the floor units, and a typical internal wall." (Building Design Partnership, 1985) However, the calculations that would justify this statement were not within the archive box of information. Without these calculations to confirm the validity of the statement it was necessary to proceed with the intrusive and non-intrusive investigations on the blocks.

On site the remedial bolting of the non-structural cladding panels back to the slab were observed in the majority of the flats. The angle is hidden behind a fire-stopping coving so is only observed if this is removed during the 'soft strip' works, hence why it was not seen in every flat. However, in every instance that the coving was removed this angle detail was revealed. As the coving is a fire-stopping measure, and the angle fixing is not implicit in the assessment for disproportionate collapse, the coving was not removed unless required for other opening up works to be carried out.

A method for fixing back the external non-loadbearing wall, thought to be an original detail, was observed in the form of a steel strap anchored with a single bolt into the top of the floor slab, see Figure 3. These steel straps were also located approximately midway along the base of the external walls in each room.

Figure 3 – Hawkinge: Strap tying base of external wall panel back to the floor slab

This steel restraint strap extends passed the end of the floor slab, running under the non-loadbearing external wall panels. The strap is tied to the wall panel by a 25mm diameter threaded bar protruding up from the wall panel below, passing through a hole in the steel strap and up into a steel 'top hat' threaded socket cast into the external wall, see Figure 4.

Figure 4 – Kenley: Treaded bar protruding from wall panel below, through hole in the strap, into a steel 'top hat' threaded socket cast into wall panel.

It is believed that an additional fixing tying the non-loadbearing external wall back to the main structure exists within the cross-wall insitu joint, and was part of the original construction of the block. BRE Report 63 shows this detail, see Figure 5, which was confirmed by the opening up works to be present in Flat 42 Kenley, see Figure 6. Being of similar construction it is, therefore, assumed that this detail is also present in all blocks.

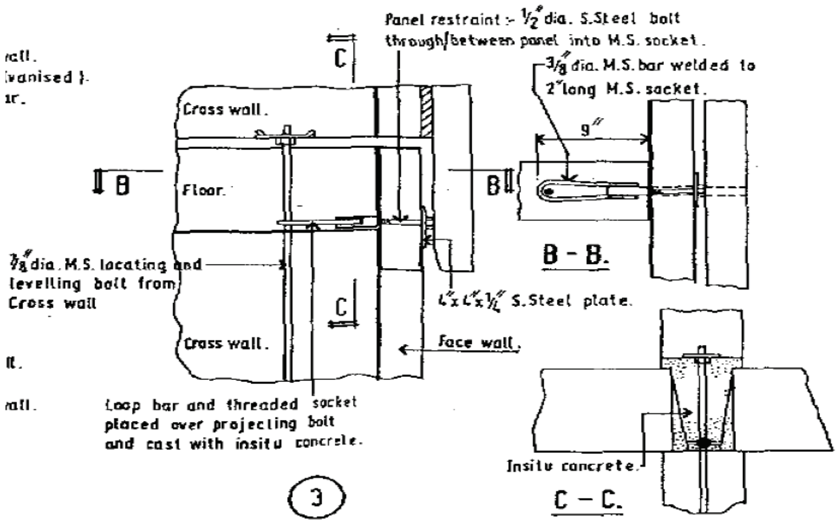

Figure 5 – Extract from BRE Report 63. Non-loadbearing cladding panel connection details. (Department of the Environment, 1985)

Figure 6 – Flat 42 Kenley - Threaded bar found within insitu cross-wall joint tying the non-loadbearing cladding panel back to the main structure

The archive information contained no construction drawings or calculations for the blocks. Subsequently Ridge requested archive searches from:

- [redacted]

[redacted]

4.3 Structural Form of the Blocks

4.3.1. 18-Storey Blocks

The two 18-storey dwelling blocks, Northolt and Kenley, are 'H-shaped' in plan with a stair core running up through the centre of the block and six flats per floor. Surrounding the stairwell is a main corridor which serves the flats. The blocks also have two lift shafts which run the full height of the block. The lifts serve alternate floors, with only one lift accessible per floor. All flats within the blocks are single storey with a floor to ceiling height of circa 2.4m.

Figure 7 – Example of 18-storey dwelling blocks (Kenley)

The two 18-storey blocks are constructed from a style of Large Panel System (LPS). It is believed that this was constructed atop an insitu concrete podium slab, supported by precast concrete beams and columns. However, in the absence of construction drawings the nature of the ground floor level construction has only been carried out by visual inspection of over-clad and hidden structural elements.

Historical reports state that these two blocks were constructed using the Larsen Nielsen System (Building Design Partnership, 1986). This form of construction involved the use of precast concrete panels, which were manufactured on the land adjacent to the site, then craned into place and assembled. Reinforcement extended from the panels into the joints between the elements and were tied together. The joints between the elements were then cast using insitu concrete and a dry-pack mortar used for final bedding of the precast loadbearing wall panel above.

The precast concrete hollow-core floor slabs were one-way spanning onto the internal cross-walls and, in the end flats, the outer flank walls. Excluding the flank walls the outer walls of the block were non-loadbearing and as such were not designed to support loading from the structure other than their own self weight and the cladding. These wall panels are stacked upon each other and tied back to the cross-walls by means of a bolt and a loop bar around the grout bar and also using steel straps which were bolted into the floor slabs. Internal concrete partition walls were also found to be present within the flats. These walls were built off the floor slabs, and do not have structural joints like the cross-walls. The internal walls are fixed back to adjacent cross/internal walls with bolts fixed into cast-in threaded sleeves in the partitions. The exterior of the block has been over-clad; this conceals the LPS nature of the block externally and the joints internally are concealed by finishes.

The floor plan for the 4th floor of Kenley can be seen in Figure 9 as an example. The floor plans for all the flats subjected to the investigations have been appended to this document for reference.

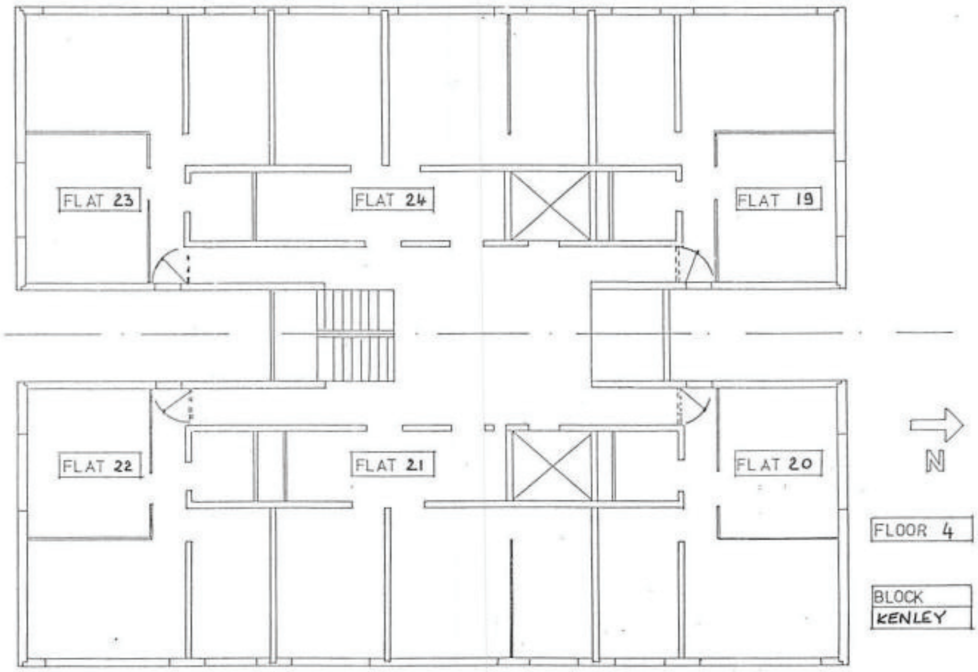

Figure 8 – Example floor plan in 18-storey dwelling block (4th Floor Kenley) (Building Design Partnership, 1985)

It is also known that although on plan the two blocks are identical, the construction details differ. Northolt was under construction before the partial collapse of Ronan Point and as such was not designed to withstand disproportionate collapse. From the historic documentation Northolt was, therefore, thought to have been retrospectively strengthened to comply with the regulations put into place following this event. Kenley, on the other hand, was we understand constructed after the event at Ronan Point and as such the construction details were amended prior to construction and should comply with the regulations for disproportionate collapse.

5. INVESTIGATION METHODOLOGY

5.1 Flats and Non-Residential Areas Subject to Investigation

The first stage of the investigative works was to carry out an initial assessment on the vacant flats and non-residential areas to determine whether they were suitable for investigation. Flats were only selected if additional information could be obtained from them, for example if the flat contained a structural element not found within other selected flats in that block.

To date, the following flats and non-residential areas were identified as suitable for investigation in the 18-storey blocks:

- Kenley:
  - Flat 42 (18-storey block, 7th floor flat)
  - Flat 16 (18-storey block, 3rd floor flat)
- Northolt:
  - Flat 55 (18-storey block, 10th floor flat)
  - Flat 41 (18-storey block, 7th floor flat)

5.2 Preparatory Works

'Soft strip' works were undertaken in the vacant flats that were identified to be suitable for the investigation, this included removal of the timber floors on top of the precast concrete floor slabs and the fire-stopping material above the windows, in order to allow a detailed visual inspection of the joints to take place.

Prior to the preparatory works commencing Homes for Haringey carried out an assessment in the flats for asbestos. Relevant precautionary measures were taken as advised by the HfH surveyor and health and safety team.

A strategy was also put in place by HfH for the eventuality where additional material potentially containing asbestos was exposed during the breaking out works. During the breaking out works the use of dust suppression was to be used, in the form of dampening down the area with water, with appropriate personal protective equipment. Should a potential source of asbestos be exposed the works were to be immediately stopped, the area to be dowsed with water and the room evacuated. Once the area was vacated the senior asbestos surveyor for HfH was to be informed. The room would then be cleared by the asbestos removal operative and subject to an environmental air test.

During the breaking out works material suspected of containing asbestos was discovered in several flats/communal areas within the "dry-pack" at the base of the wall panels. The agreed procedure was followed in each instance.

5.3 Non-Intrusive Ferro Scan Works

The load bearing walls within each of the flats were identified. In general, the walls with the least services (pipes/wires etc.) were selected for scanning works.

The critical areas, in terms of assessing the blocks against disproportionate collapse, are around the insitu joints between the main structural elements. As such, the non-intrusive scanning works were focussed on the cross-walls, flank walls, and floor slabs (and where applicable the corbels and R.C. beams) close to the intersection of these members.

The scanning works were carried out on 600mm x 600mm grids using a HILTI PS250 Ferro-magnetic scanner. The data was then transferred onto a HILTI PSA 100 tablet for viewing and review of the scans. Examples of the scan data can be seen in Figure 9 and Figure 10.

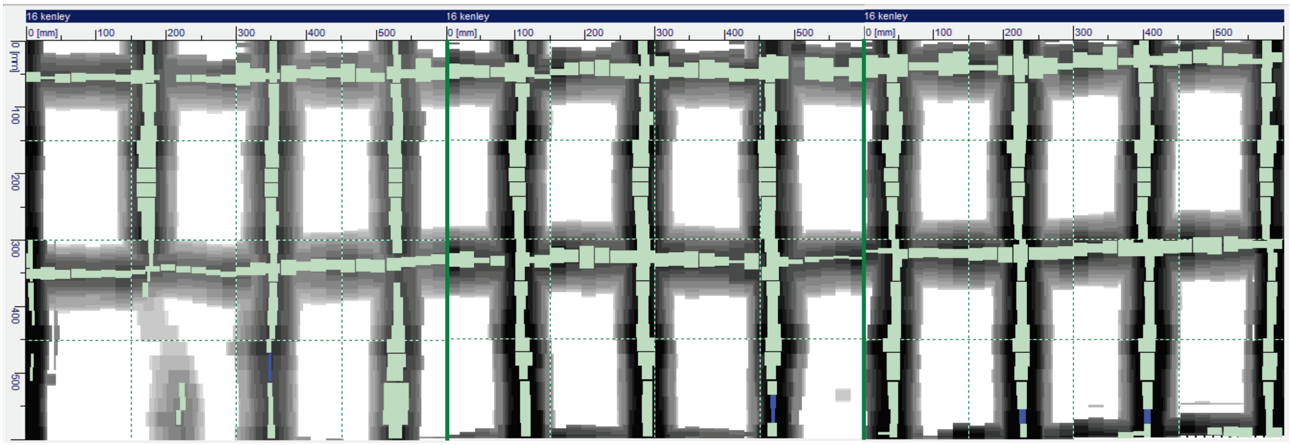

Figure 9 – Example of Three Wall Scans taken in Flat 16, Kenley. The scans clearly show bars at regular 150mm c/c with two clear lateral bars extending the full width of the three scans.

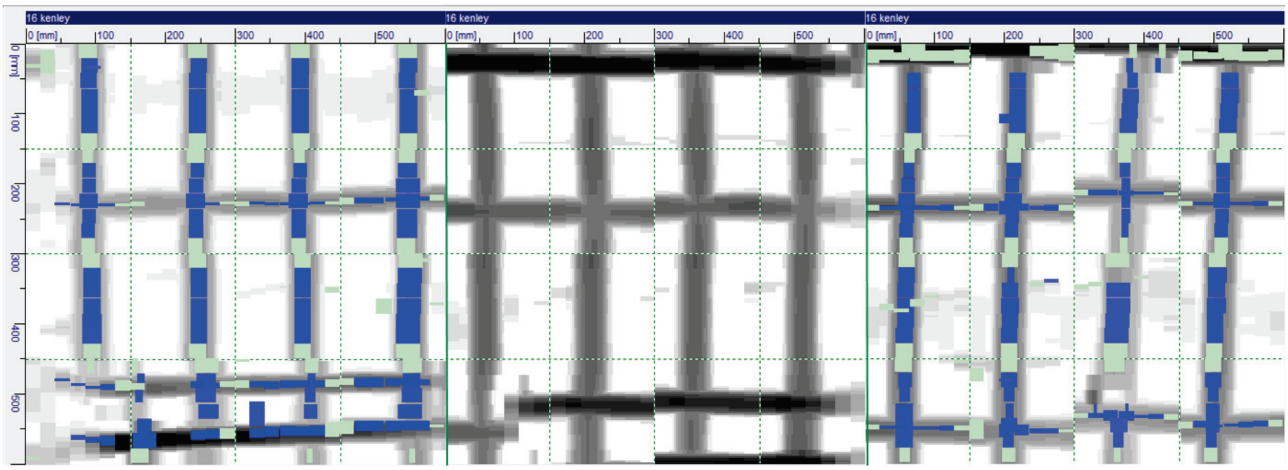

Figure 10 – Example of Ceiling Scan Data taken in Flat 16, Kenley. Scan shows bars at close regular centres at the base of the slab. It should also be noted the difference in clarity between the bars in scan 2 compared to scans 1 & 3. This was a common occurrence throughout the scanning with some bars being harder to identify.

The HILTI PS250 was then set to the cover-meter function and the walls and slabs re-scanned to determine the cover to the reinforcement bars (rebar). The location and cover of the rebar was marked up onto the walls. A detailed interpretation of the information obtained through these two non-invasive investigation methods was undertaken to identify areas required for further investigation and breaking out works.

5.4 Interpretation of Non-Intrusive Investigation Results

The results from the Ferro scan mapping and the cover-meter mark-up on the walls were then compared. The combination of the two sets of data enabled the engineers to identify the areas known to be critical in the investigation of structural adequacy for disproportionate collapse. In particular, the location around the 25mm link bars typically provide the information required to determine whether the structure possessed effective vertical and horizontal ties.

Once the areas of interest were identified the engineers marked-up the walls to indicate to the contractor where the intrusive opening up of localised pockets was to be carried out.

An example of the walls marked-up with the information obtained from the two sets of scans can be seen in Figure 11.

Figure 11 – Wall of Flat 42, Kenley marked-up with the results from the two scans. The information obtained then used to identify areas for further investigation using intrusive methods.

5.5 Intrusive Opening-Up Works

For the intrusive investigation Ridge arranged for a contractor to carry out the opening-up works under the supervision of an engineer. Having previously identified the areas of interest to the investigation using the non-intrusive methods the contractors carefully broke out pockets in the concrete, where required, using HILTI electromechanical concrete breakers.

Once broken out, the pockets were visually inspected by the engineer to validate the results from the Ferro Scanning works and to understand the reinforcement details and ultimately the construction of the block.

These details were measured and recorded to enable calculations to be produced to assess the adequacy of the elements to resist accidental over-pressure loading.

Initially, several pockets were made in each flat to aid in the understand of how the blocks were constructed. During the investigation many similarities were found between the blocks in the way the panels were connected within the insitu joints. These similarities were observable in the scanning data. This allowed the amount of intrusive opening up works to be progressively scaled back. Intrusive opening up works were only carried out in the later flats to verify any discrepancies found within the scan data.

6. FINDINGS OF INVESTIGATION

6.1 Overview

A combination of intrusive and non-intrusive investigations has been carried out within select flats in blocks assessed to date. The aim of this section is to compare the results of similar blocks to find common details, and to highlight any discrepancies.

6.2 Rebar Grades

Assessing the age of the structures it is likely that the rebar within all twelve of the dwelling blocks conformed with BS 1478:1964. The three rebar types noted in this code are:

- MR = Round Mild Steel
- HR = Round High Yield
- HS = Square High Yield

Opening up works of the precast units and the associated joints showed that plain round bars, ribbed bars, and twisted square bars were used in the construction of the block. As it was not possible to determine whether the plain round bars were mild steel or high yield bars without record information this report will consider them to be type 'MR' with a characteristic yield strength of 250MPa. The ribbed bars were 'HR' type bars with a characteristic yield strength of 450MPa. The twisted square bars were likely 'HS' type bars with a characteristic yield strength of around 450MPa.

6.3 Overview of findings within 18-Storey Blocks

The investigative works have shown that the design and construction of the two blocks are substantially different. Although, this was to be expected due to the reported design revisions after the partial collapse of Ronan Point.

<u>Kenley</u>

The construction of Kenley generally comprised of 8" thick hollow core slabs spanning between 7" centrally reinforced pre-cast cross-walls with vertical link bars at approximately 4' centres. The flank walls are a 6"reinforced concrete panel with a single layer of reinforcement toward the rear of the panel, 25mm polystyrene insulation and an external 75mm concrete panel, link bars were found at 2' centres. The cladding to the non-load bearing east and west elevations is vertically stacked and restrained by the structure only at each level.

<u>Northolt</u>

Northolt generally comprised of 8" thick hollow core slabs spanning between 7" largely unreinforced pre-cast cross-walls. The cross-wall panels contained link bars at both ends of the panel, but no intermediates. The flank walls are 6" unreinforced concrete panel with 25mm polystyrene insulation and an external 75mm concrete panel. Again, the link bars were found at the ends of the panels, but no intermediate link bars were found.

Remedial steel angles were noted bolted through cross-wall and floor slab at ceiling level. No angles were noted to the base of the cross-walls, the bolts from angles below bolt onto 2-½ '' square washer plate on top of the floor slab.

The flank wall to floor slab joints were noted to have had remedial strengthening carried out. Steel angles had been bolted through flank wall and floor slab at base and head of wall panel. A threaded sleeve and been retrospectively bonded into core in flank wall panel to accept the bolt.

The cladding to the non-load bearing east and west elevations is vertically stacked and restrained by the structure only at each level.

7. LIMITATIONS OF INVESTIGATION

As part of the investigation into the robustness of the block of flats at Broadwater Farm Estate Ridge carried out a review of the historic information held by Homes for Haringey. It was noted that much of the information held was architectural and contained no construction drawings or calculations for the blocks. Ridge also requested archive searches from the original contractor, consulting engineer and Hornsey Town Hall (known to have held the drawings historically). However, at time of writing no additional information has been provided. In the absence of the construction details more breaking out work was required to understand the construction of the building before the robustness assessment could take place.

The investigation was also affected by the presence of asbestos in the ceiling coatings and within parts of the structure. This caused delays to the project timescales as this had to be stripped out prior to the breaking out works.

Whilst the investigation on each block was detailed, with multiple wall and floor scans and several subsequent pockets opened up, it should be noted that there is the possibility for lack of continuity between the joints in the same block. As the investigations were only carried out in two of the flats per block there is the possibility that other joints, not included in this investigation may contain imperfections, may be a product of poor workmanship, have missing bars, damaged/corroded rebar etc. Without checking every joint in the block it is impossible to provide 100% certainty that all joints have been constructed correctly. A full assessment of every joint in the blocks would not be practical and thus the assessment of the blocks can only be based on what was uncovered in the sample investigation.

8. ASSESSMENT

8.1 Classification of Structure

The Building Regulations 2010 Approved Document A divides buildings into consequence classes depending on their purpose and number of storeys using Table 1.

Consequence Classes	Building Type and Occupancy
1	Houses not exceeding 4 storeys
	Agricultural buildings
	Buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of 1.5 times the building height
2a Lower Risk Group	5 storey single occupancy houses
	Hotels not exceeding 4 storeys
	Flats, apartments and other residential buildings not exceeding 4 storeys
	Offices not exceeding 4 storeys
	Industrial buildings not exceeding 3 storeys
	Retailing premises not exceeding 3 storeys of less than 2000m^2 floor area in each storey
	Single-storey educational buildings
	All buildings to which members of the public are admitted which contain floor areas exceeding 2000m^2 but less than 5000m^2 at each storey
	Car parking not exceeding 6 storeys
2b Upper Risk Group	Hotels, blocks of flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys
	Educational buildings greater than 1 storey but not exceeding 15 storeys
	Retailing premises greater than 3 storeys but not exceeding 15 storeys
	Hospitals not exceeding 3 storeys
	Offices greater than 4 storeys but not exceeding 15 storeys
	All buildings to which members of the public are admitted which contain floor areas exceeding 2000m^2 but less than 5000m^2 at each storey
	Car parking not exceeding 6 storeys
3	All buildings defined above as Consequence Class 2a and 2b that exceed the limits on area and/or number of storeys
	Grandstands accommodating more than 5000 spectators
	Buildings containing hazardous substances and/or processes

Table 1 - Disproportionate Collapse Consequence Classes (Department for Communities and Local Government, 2010)

The two blocks assessed in this report, Kenley and Northolt, are both eighteen stories above what is currently understood to be an insitu concrete ground floor service area. These blocks are, therefore, within consequence class 3 '*All buildings defined above as Consequence Class 2a and 2b that exceed the limits on area and/or number of storeys*'

Attached to the main six-storey blocks are four-storey wings. These wings are within the 2a (lower risk group) consequence class '*Flats, apartments and other residential buildings not exceeding 4 storeys'*

8.2 Assessment Criteria

BRE Report 511 – Handbook for the Structural Assessment of Large Panel System (LPS) Dwelling Blocks for Accidental Loading provides guidance on how to determine whether an LPS block complies with the

Building Regulations for disproportionate collapse. The report identifies three criteria for assessment, of which the blocks need only pass one:

- 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 (see Section 8.1) 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. (Matthews & Reeves, 2012)

8.3 Assessment Discussion – 18 Storey Block – Kenley

From Approved Document A, Kenley is defined as a Consequence Class 3 Structure. Therefore, the building requires effective horizontal and vertical ties within the connections and a systematic risk assessment, taking into account normal and abnormal hazards that can reasonably be foreseen.

This therefore means to meet disproportionate collapse requirements LPS Criterion 2 must be assessed as part of the systematic risk assessment, regardless of the results of LPS Criterion 1 and 3.

8.3.1. LPS Criterion 1

Assessment of the cross-wall and flank wall joints exposed that there were 10mm diameter U-bars at 12" centres protruding from the edges of the precast floor slabs. The U-bars from the adjacent floor slabs were joined within the joint with 2no. 12mm diameter lacer bars. This connection constituted the horizontal tie in the joint. This connection method is highlighted in MHLG Circular 62/68 as an acceptable horizontal tie connection.

The vertical tie in the cross-wall joints consisted of 20mm bars at 4' centres cast into the top of each wall panel extending up through the insitu joint into a grout pocket in the bottom of the panel above. The investigative works have also shown that these bars are not continuous in the wall panels and extend circa 500mm up into a grout pocket. This connection method is not sufficient to provide an effective vertical tensile tie. In the event of a failure of a cross-wall, the floor above which would have originally been taking bearing on the failed wall, would be reliant on hanging from the vertical bars protruding down from the wall above. These plain round bars are grouted into the walls and would offer little tensile resistance due to the minimal bond a smooth bar achieves. A pull-out failure of the bars would therefore be likely, causing the floor to fall away from the wall above.

The vertical tie in the flank wall joints consisted of 20mm bars at 2' centres cast into the top of each wall panel extending up through the insitu joint into a grout pocket in the bottom of the panel above. These bars were also found to extend circa 500mm up into a grout pocket. Due to the reduced spacings of these link bars the connection has been shown to be an effective vertical tie. The flank wall connection, therefore, passes LPS Criterion 1.

Other issues regarding the joints between connection between loadbearing elements were also highlighted in the hallway of Flat 16.

A downstand beam spans across the hallway onto the external flank wall panel. However, the interface between the downstand and the flank wall had no insitu joint and there was no connection between the two elements. The flank wall was cast with a formed pocket at the top, onto which the downstand took at 1'' bearing. This is not an adequate connection in modern terms.

A further issue noted was the connection of the floor slabs with the downstand beam. Although there was an insitu joint above the downstand only the two floor slabs had any connection. Nothing was found that would indicate that the downstand was connected to either floor slab. This is also not an adequate connection.

As not all of the joints provide an effective vertical tie it is deemed that Kenley does not comply with LPS Criterion 1.

8.3.2. LPS Criterion 2

Kenley does not have a piped-gas supply, and as such the blocks are subject to the reduced over-pressure of 17kN/m^2.

The floor slabs had reinforcement in the top and bottom of the slab. The 'key element' calculations checked the robustness of the floor slab to resist the 17kN/m^2 over-pressure. This analysis showed that the floor construction was sufficiently robust.

The cross-wall panels were found to be centrally reinforced, with additional 20mm link bars at 4' centres extending from the insitu joint into the wall panel. One of the failure mechanisms identified for the cross-walls was local shear splitting failure around these link bars. The analysis calculations showed that the cross-walls would not fail under this form of failure in the event of an internal explosion with an over-pressure of 17kN/m^2. The cross-walls were also analysed in flexure for the 17kN/m^2 over-pressure. The calculations showed that the reinforcement provided within the panel was sufficient to resist the loading.

The main reinforcement in the flank wall panels was found to be placed near the outer face of the panel. The additional 20mm link bars which extended from the insitu joint into the wall panel were found to be placed at a reduced spacing of 2'. One of the failure mechanism identified for the flank walls was local shear splitting failure around these link bars. However, as the link bars were found to be at smaller centres than the cross-wall (which passed the analysis for this failure mechanism) the flank walls were considered to be sufficient to resist this failure mechanism under the associated over-pressure. The flank wall was also analysed in flexure for the 17kN/m^2 over-pressure. The calculations showed that the reinforcement provided within the panel was sufficient to resist the loading.

8.3.3. LPS Criterion 3

It has been shown in the key element check calculations that Kenley passes the Criterion 2 assessment. Therefore, LPS Criterion 3 need not be carried out for this block.

8.3.4. Systematic Risk Assessment

In the context of new build high rise design, the building regulations requires a systematic risk assessment to be undertaken for (new build) class 3 structures such as this. Whilst this is not a retrospective requirement it is good practice to consider risks to disproportionate collapse, such as vehicular impact damage which could then be mitigated using vehicular barriers for example. A draft risk assessment for both blocks has been provided under separate cover.

Following the assessment of Kenley the block passes LPS criterion 2 with the exception of the downstand beam which will require remedial strengthening works for the block to fully pass this criteria.

8.4 Assessment Discussion – 18 Storey Block – Northolt

From Approved Document A, Northolt is defined as a Consequence Class 3 Structure. Therefore, the building requires effective horizontal and vertical ties within the connections and a systematic risk assessment, taking into account normal and abnormal hazards that can reasonably be foreseen.

This therefore means to meet disproportionate collapse requirements LPS Criterion 2 must be assessed as part of the systematic risk assessment, regardless of the results of LPS Criterion 1 and 3.

8.4.1. LPS Criterion 1

Assessment of the cross-wall and flank wall joints exposed that, unlike Kenley, there were no U-bars protruding from the edges of the precast floor slabs, nor were there any other forms of connection within the insitu joint. This was to be expected as the block was under construction before the partial collapse of Ronan Point, the 'turning-point' event leading to the introduction of regulations for disproportionate collapse.

Instead the joints were subject to remedial strengthening works in the form of steel angles bolted to the loadbearing elements and their intersections.

The flank wall connection is formed of two steel angles, one at ceiling level and the other at floor level. The two angles are bolted together, through the floor slab, by M16 bolts at max. 750mm centres. The angles are also bolted to the flank wall panels by M16 bolts at max. 900mm centres inside metal sleeves bonded to the concrete.

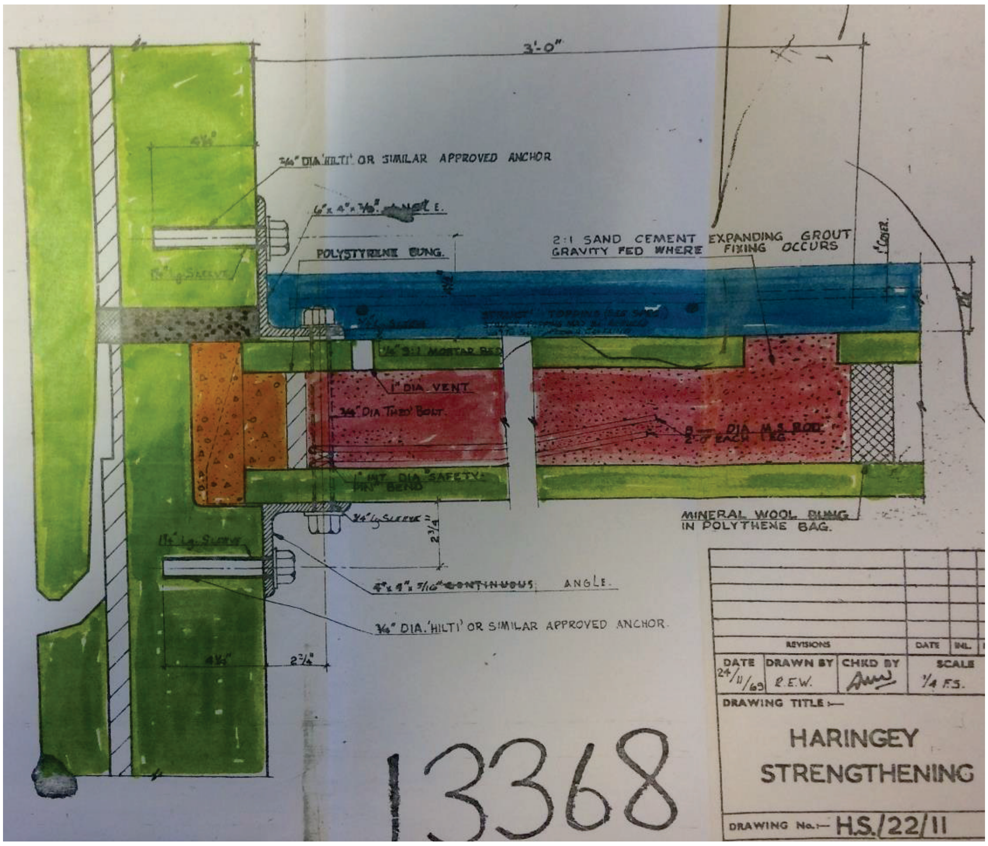

Figure 12 – Historic strengthening detail for flank wall joint in Northolt (Phillips Consultants LTD, 1969)

Intrusive investigative works were carried out around the bolts through the floor slab. This area was opened up in both Flat 41 and Flat 55. In both cases the 8mm U-bar, shown in the historic detail to be cast into the grout infilling the core and looping around the bolt had not been installed, as per the design. In both instances the entire core was exposed.

Pockets were also created in the flank wall panel around the sleeved anchors. The intrusive works showed that the total embedment of the anchor was circa 100mm. The metal sleeve was also exposed to be smooth, with little adhesion to the concrete panel which was demonstrated by the concrete breaking away cleanly from the sleeve. Figure 13 shows a pocket around one of these fixings.

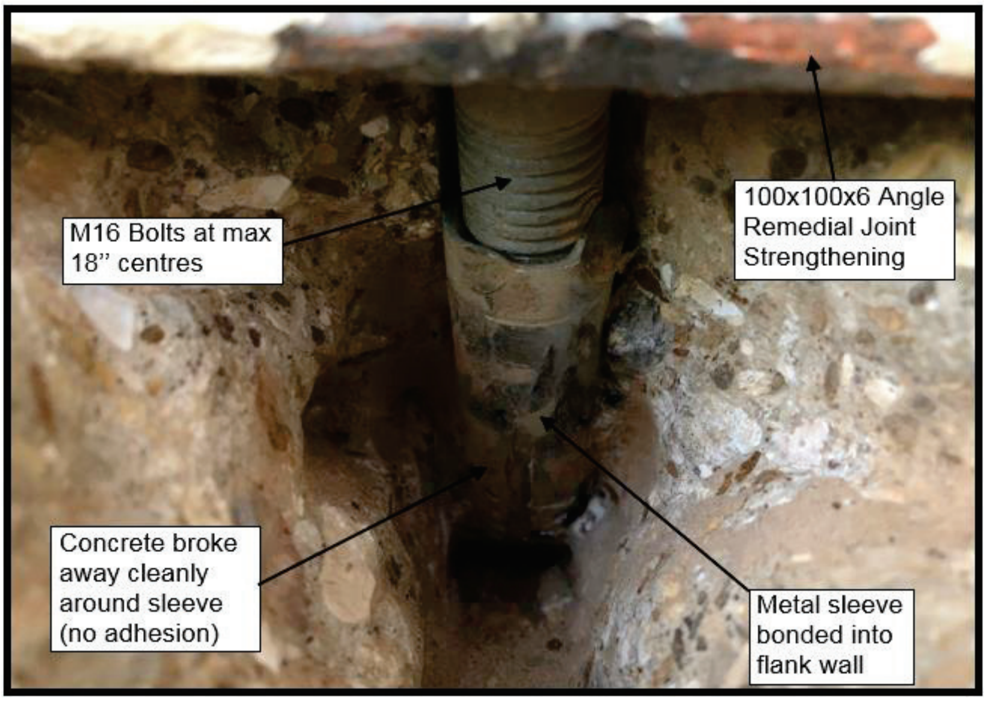

Figure 13 – Photo-log of pocket around sleeved anchor into flank wall panel.

In the absence of details of the actual fixings used during the strengthening works in the 1960's the assessment on the adequacy of this connection was carried out based on the most similar fixing in Hilti's current range. The Hilti HIS-N internally threaded sleeve and HVU Adhesive was selected for the analysis.

Several key differences were noted between the fixings installed in Northolt and the HIS-N anchors:

- The HIS-N Sleeve is ribbed, whereas the actual sleeve is smooth;
- The HIS-N Sleeve requires a minimum embedment of 170mm, whereas the actual embedment is 100mm;
- The HIS-N Sleeve requires a minimum edge distance of 90mm, whereas the actual edge distance is 75mm;
- HVU Adhesive will be greatly advanced compared to the actual substance used to bond the sleeves to the flank wall panel, which is providing little to no adhesion.

The HIS-N anchors were shown to be sufficient to resist failure for the over-pressure of 17kN/m^2. However, the differences between the HIS-N anchors to the actual fixings used for the strengthening works were deemed significant. The installed fixings are therefore likely to be inadequate to resist the over-pressure loading, thus not satisfying LPS Criterion 1.

The cross-wall joint is formed of two steel angles, both at ceiling level, bolted together through the cross-wall at max. 750mm centres. The angles are also bolted, through the floor slab, at max 750mm centres to 65mm square washer plates. There was no connection to the base of the cross-wall panel.

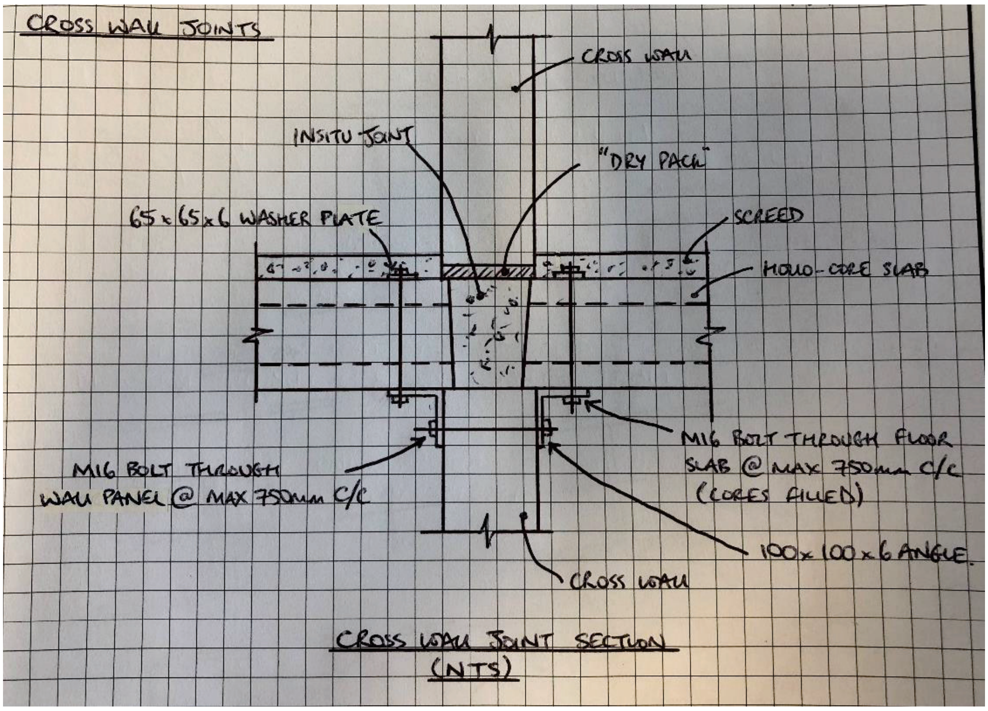

Figure 14 – Indicative cross-wall joint detail from investigative works

As there is no connection to the base of the cross-wall panel there the connection does not possess an effective vertical tie. Therefore, the connection does not satisfy LPS Criterion 1.

Other issues regarding the joints between connection between loadbearing elements were also highlighted in the hallway of Flat 16.

A downstand beam spans across the hallway onto the external flank wall panel. However, the interface between the downstand and the flank wall had no insitu joint and there was no connection between the two elements. The flank wall was cast with a formed pocket at the top, onto which the downstand took at 1'' bearing. This is not an adequate connection.

8.4.1. LPS Criterion 2

The two eighteen storey blocks do not have a piped-gas supply. As such, the blocks are subject to the reduced over-pressure of 17kN/m^2.

The floor slabs had reinforcement in the bottom of the slab but was unreinforced in the top. The 'key element' calculations checked the robustness of the floor slab to resist the 17kN/m^2 over-pressure. This analysis showed that the floor construction was insufficiently robust under both downwards and upwards loading.

The cross-wall panels were found to be largely unreinforced. The cross-walls contained 20mm link bars at each end of the panels but, unlike Kenley, there were no intermediate link bars. As the wall panels were largely unreinforced one of the failure mechanisms identified was flexural failure. The analysis calculations utilised the tensile strength of the concrete and showed that the cross-walls in floors 8-17 would fail under this failure mode in the event of an internal explosion with an over-pressure of 17kN/m^2. The cross-walls in the lower floors were shown to pass the assessment due to the increased compressive stresses in these units.

The flank wall panels were also found to be largely unreinforced. The cross-walls contained 20mm link bars at each end of the panels but, unlike Kenley, there were no intermediate link bars. As the wall panels were largely unreinforced one of the failure mechanisms identified was flexural failure. The analysis calculations showed that the flank walls on every floor failed the checks for this failure mode in the event of an internal explosion with an over-pressure of $17kN/m^2$.

8.4.2. LPS Criterion 3

For a block to pass Criterion 3 the structure must be able to mobilise alternative load paths in the event of an explosion. As the blocks do not have a piped-gas supply the boundary enclosure is deemed to be a single room. In the case of the living room of the end flats the room is enclosed by a flank wall and a cross-wall. If an explosion was to occur within this space two of the loadbearing walls would fail, along with two of the floor slabs, as shown in the assessment carried out for LPS Criterion 2. In this instance no alternative loadpaths could be mobilised, potentially resulting in a disproportionate collapse event for the block as all support in removed or compromised.

8.4.3. Systematic Risk Assessment

In the context of new build high rise design, the building regulations requires a systematic risk assessment to be undertaken for class 3 structures such as this. Whilst this is not a retrospective requirement it is good practice to consider risks to disproportionate collapse, such as vehicular impact damage which could then be mitigated using vehicular barriers for example. A draft risk assessment for both blocks has been provided under separate cover.

From the appraisal carried out in accordance with the BRE report 511 Northolt does not pass the LPS criteria, therefore we would recommend that a study is undertaken that looks at strengthening options for the block in order to pass a minimum over-pressure of $17kN/m^2$.

9. FURTHER OBSERVATIONS

During the investigation into the robustness an observation was noted that the reinforcement found within the joints does not effectively deal with the stresses known to be present in joints of this construction. The forces and stresses present in the joints can be seen in the extract from fib Bulletin 43 in Figure 15.

For example, U-bar reinforcement is normally found to the bottom of the wall panels, the base of the wall panels investigated did not have such reinforcement. However, from our observations on site there were no obvious signs of structural distress from the lack of the appropriate joint reinforcement, this is most likely due to the low stresses contained within the joints of the current structural configuration.

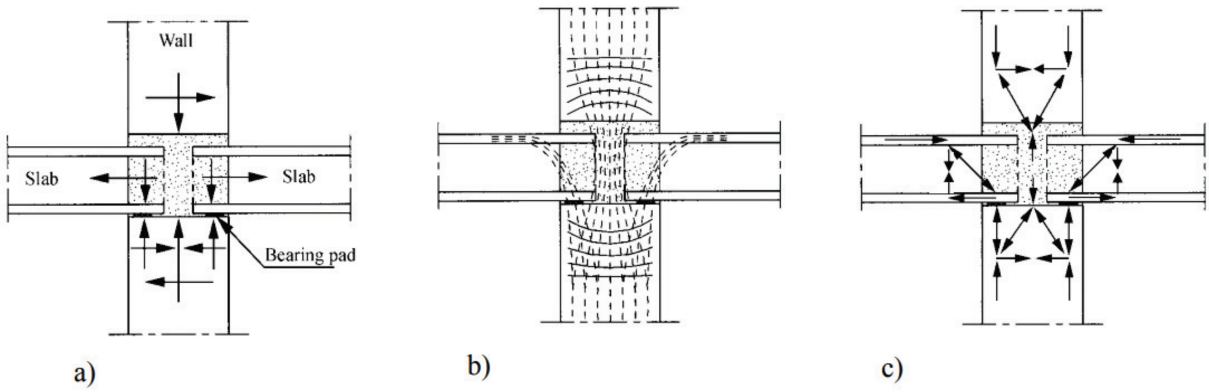

Figure 15 – Slab-wall connections, a) forces, b) simplified stress analysis, c) strut-and-tie model (fib, 2008)

Throughout the investigation cracks were found to be a common occurrence in the loadbearing wall panels. The majority of cracks were vertical cracks running the entire height of the wall, and all the way through the thickness of the panel. There are a number of likely causes for the cracking within the panel which are thermal, shrinkage, poor curing or construction related damage. The cracks observed are not thought to affect the load bearing adequacy of the structure.

Another observation was that the non-loadbearing external wall panels are vertically stacked, and fixed back to the main structure primarily to reduce lateral deflections in the panels. In the event of an explosion in one of the flats the non-loadbearing external wall panel, and its fixings back to the structure, would be insufficient to resist the 17kN/m^2 over-pressure. This would cause the panels to ‘blow out’ and fall away from the structure.

As these panels are vertically stacked if the explosion were to occur in the lower floors this may result in a ‘domino effect’, with the panels above progressively losing their support and potentially falling to the ground. Although this would not be deemed to be disproportionate collapse as no floor area would be at risk of collapse with the removal of these panels (unless struck by the falling debris) there is the potential to cause considerable collateral damage, injury or loss of life.

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