LONDON BOROUGH OF HAMMERSMITH & FULHAM

HARTOPP & LANNOY HIGHRISE BUILDINGS

Structural Assessment of Hartopp and Lannoy Highrise Buildings Aintree Estate

(Large Panel System Buildings)

STAGE (II) REPORT

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HARTOPP POINT

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1.0 Executive Summary

The London Borough of Hammersmith & Fulham Building Control Structural Engineers were commissioned to carry out a structural assessment of the two tower blocks in Fulham known as Lannoy Point and Hartopp Point to assess their resistance to disproportionate collapse in the event of an accidental explosion and their performance under normal dead, live and wind loading.

In order to comply with the brief an intrusive investigation into the fabric of the buildings was carried out to confirm the construction and condition of the towers.

The findings are summarized as follows:

i. Structurally, both Lannoy and Hartopp high-rise buildings appear to have performed satisfactorily over their service period of over 45 years. No signs of historic or newly occurring distress were observed under the service dead and live loads in terms of fractured wall and floor panels. Cracks in mortar filled joints were observed in places. No wall panels have been reported dislodged or moved out of position in the 45 years of service suggesting that wind loading on the buildings have been accommodated and resisted without signs of distress.

ii. Based on the information obtained from the intrusive survey, it has been concluded that the buildings were not designed to accommodate damage and prevent disproportionate collapse in the event of an accidental explosion. The construction is therefore non-compliant with the requirements for Class 2B buildings as set out in Approved Document (AD) A (Structure) to the Building Regulations 2010 (as amended).

iii. Both Lannoy and Hartopp high rises were built around the same time as Ronan Point which is around 1968 or 1969.

A solution has been proposed to bring about compliance with the requirements of class 2B buildings as set out in Approved Document [AD] A. This solution requires the strengthening of the tower blocks. The indicative scheme is presented in Appendix A attached.

With regards to the management of the tower blocks in the future, it is recommended that bottled gas and oxygen cylinders should be prevented from entry and storage in the buildings. For further information on the management of these risks and the assessment approach, Appendix C of the Handbook for the structural assessment of Large Panel Systems (LPS) dwelling blocks for accidental loading is recommended reference.

2.0 Introduction

This report is the stage (II) report which addresses the issue of structural robustness in the two high-rise buildings and any defects in line with the recommendations laid out in BR511[1] for large panel systems generally referred to as LPS. The stage (II) survey commenced in September and concluded in December of 2017.

The survey was both visual and intrusive. Particular attention was paid to the issue of the 'disproportionate collapse prevention' design, or the lack of it, and the robustness of the construction of the two tower blocks.

The LPS dwelling block system type
The precast concrete Large Panel System (LPS) used in the construction of Lannoy and Hartopp high-rise blocks is the Larsen Neilsen System, a Danish method of building from large concrete panels. The system was modified in the 1970's adding wall ties. This system was later bought out by Taylor Woodrow Anglian. The Larsen-Nielson system was composed of factory-built, precast concrete components designed to minimize on-site construction work. Walls, floors and stairways are all pre-cast. All units, installed one-story high, are load bearing (System, 1968). This building technique encompassed the patterns for the panels and joints, the method of panel assembly, and the methods of production of the panels. In this type of structural system, each floor was supported by the load bearing walls directly beneath it. Gravity load transfer occurred only through these load-bearing walls. This wall and floor system fitted together in slots. These joints were then bolted together and filled with insitu concrete / dry pack mortar to secure the connection.

The intrusive investigation into the tower blocks uncovered no vertical wall ties as was expected of the system type. The only bolted wall panel connections observed were to the lift and stair access tower (Figures 4, 5 & 6). The bolts replace the reinforcement anchoring of the concrete panels together to provide the connectivity.

The conclusion of the stage (II) investigation is that the tower blocks were not compliant with the requirements of class 2B buildings as laid out in Approved Document A to the Building Regulations and an indicative remedial scheme for the strengthening of the blocks has been added to the report in Appendix A.

3.0 Building Description & Structural Form

3.1 Building Description

[Figure 1] Elevations of the High-Rise Buildings

Hartopp and Lannoy tower blocks are two fourteen-story residential precast concrete LPS construction buildings built in c1969. Both buildings are identical in construction consisting of two identical residential towers connected by link bridges to a central access tower. This access tower comprises two lifts and a stair core. The stair core is an extra story higher to accommodate a lift mortar room. The cladding finish is exposed aggregate concrete panels and the foundations are presumed to be piled.

Both tower blocks have 56 flats each. Ground to fourth floor consists of 3 & 1-bedroom flats per wing per level and the remainder is all 2 bed flats per wing per level. The stair core serves both wings and there are two lifts serving alternate floors, however they terminate at level 12.

[Figure 2] Plan Layout of the High Rise Buildings

3.2 Structural Form

The two tower blocks are basically a stack of precast concrete walls and floor slabs. Unlike modern concrete tower blocks which are designed robustly with a skeletal structure comprising insitu concrete lift and stair cores, shear walls and flat slab floors, all tied together with reinforcement to accommodate accidental damage, Hartopp and Lannoy towers were not designed to limit disproportionate collapse in the event of an accidental damage.

The external walls are sandwich panels with insulation in the middle, whilst the internal walls are combination of solid concrete panels and timber partitions. The internal concrete cross walls and flank walls provide lateral stability to the towers and the concrete floors provide the diaphragm action to transfer the wind shear.

The floor slab is 190mm thick precast concrete hollow core panels with 40mm screed.

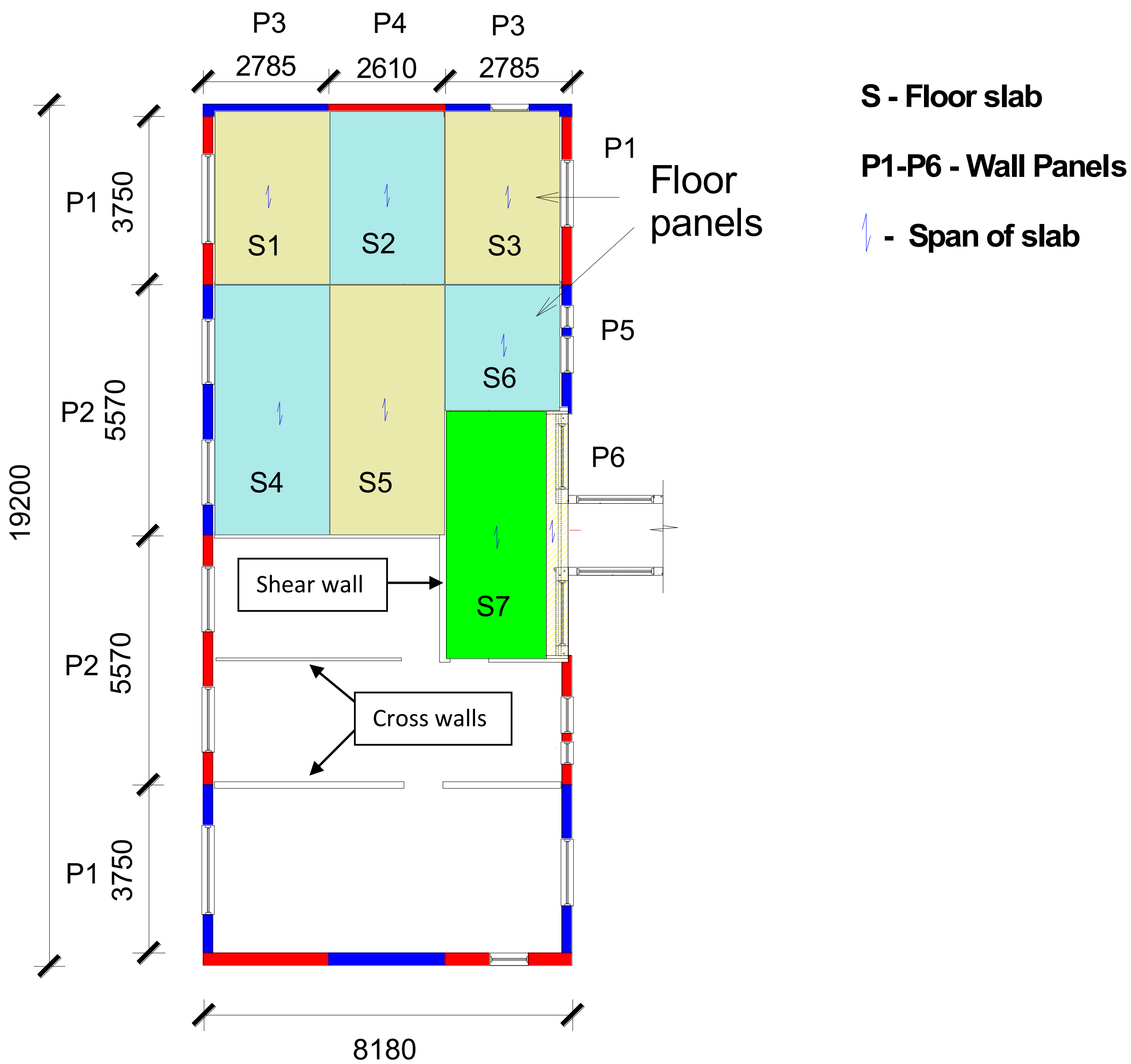
It is presumed that the foundations are piled foundations with concrete caps and ground beams. The layout and specific design of the foundations are unknown. No intrusive investigation into the foundations was carried out.

Figures 3a and 3b show the survey information. The floor panels appear to be of four different sizes and span in one direction onto load bearing concrete walls. The wall panels are marked 'P' on the floor plan. Flank wall panels P3 and P4 are the only wall panels vertically connected by internal reinforcement u-bars and dowels.

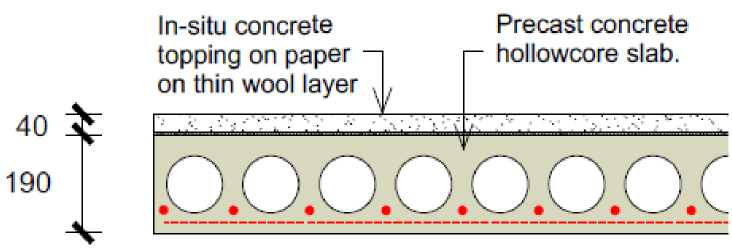
Wall panels P1 and P2 have no vertical interconnecting reinforcement ties but are held in place be tie-back steel plates (Figures 13, 14, 15, 16). Wall panel P2 is a double panel single piece wall unit spanning two rooms.

Summary of wall and floor panel types [Table 1]

Item	Description
Precast Concrete Floor [S1 to S7]	These are 190mm thick precast concrete hollow core slab + 40mm screed simply supported on concrete walls. Cores presumed filled in at ends to facilitate bolting of the steel angles. Joint gaps were grouted.
External Flank Wall Panels [P3 & P4]	280mm thick sandwich load bearing concrete panel with 165mm inner skin. All three individual flank wall units are stitched together vertically using 'u' bars and lacer.
External Side Wall Panels [P1, P2, P5 & P6 Window Panels]	215mm thick sandwich load bearing panel.
Internal wall [type 1]	235mm thick solid precast shear wall panel with inbuilt tie bars for tension stability. Vertically continuous stability element to tower blocks.
Internal wall [type 2]	150mm thick solid precast concrete panel.
Internal wall [type 3]	65mm thick solid precast concrete partition panel.
Internal wall [type 4]	65mm thick timber internal partition.
Core wall panel [type 5]	180 mm thick load bearing external walls to access core.
Bridge wall panel [type 6]	180 mm overall thick non-load bearing external walls to Link Bridge.



[Figure 3a] Floor & Wall Panel Layout Plan



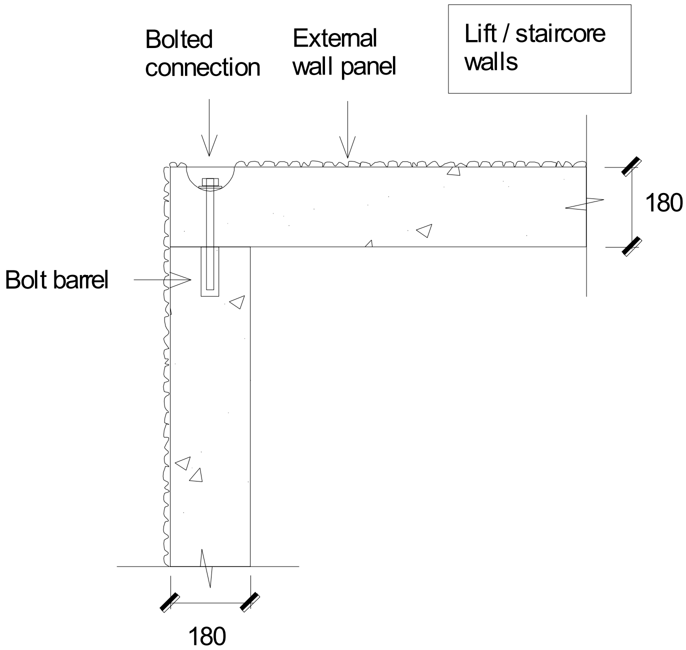
[Figure 3b] Floor Construction

3.2.1 Lift and Stair Core

The lift and stair access core is the central core located between the two residential wings. The external walls are 180mm thick solid concrete panels. The stairs are precast concrete units connected to the landing units. The external walls have bolted connections. The link bridge walls are 180mm thick solid concrete panels tied into the slab at floor level with straps and dowels.

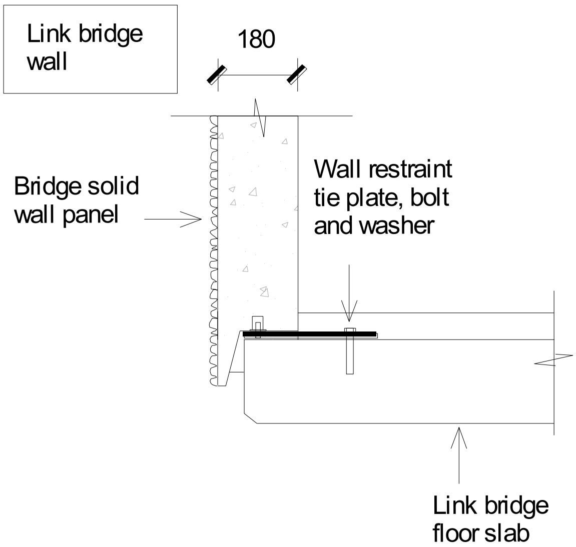
[Figure 4] Stair core bolted connection

[Figure 5] Stair core bolted connection exposed.



[Figure 6] Stair core bolted connection detail.

3.2.2 Link Bridge



[Figure 7] Section through Link Bridge at first floor.

[Figure 8] First floor Link Bridge.

4.0 Assessment Stage 1: Review of Existing Information

No existing construction drawings were found to assist the stage (II) survey and report. Measured survey was then carried out in order to create existing plan drawings and section details for this report. No structural calculations were found pertaining to the strengthening works neither was there found any documentation pertaining to any past or ongoing inspection regime.

5.0 Assessment Stage 2: Collection of New Technical Information

5.1 Areas Surveyed

The intrusive survey was carried out in the following places:

- Lannoy Point – Flats 1 & 27.
- Hartopp Point – Flat 31.
- Hartopp Point levels 12 & 13 link bridges.
- Hartopp Point stair core.

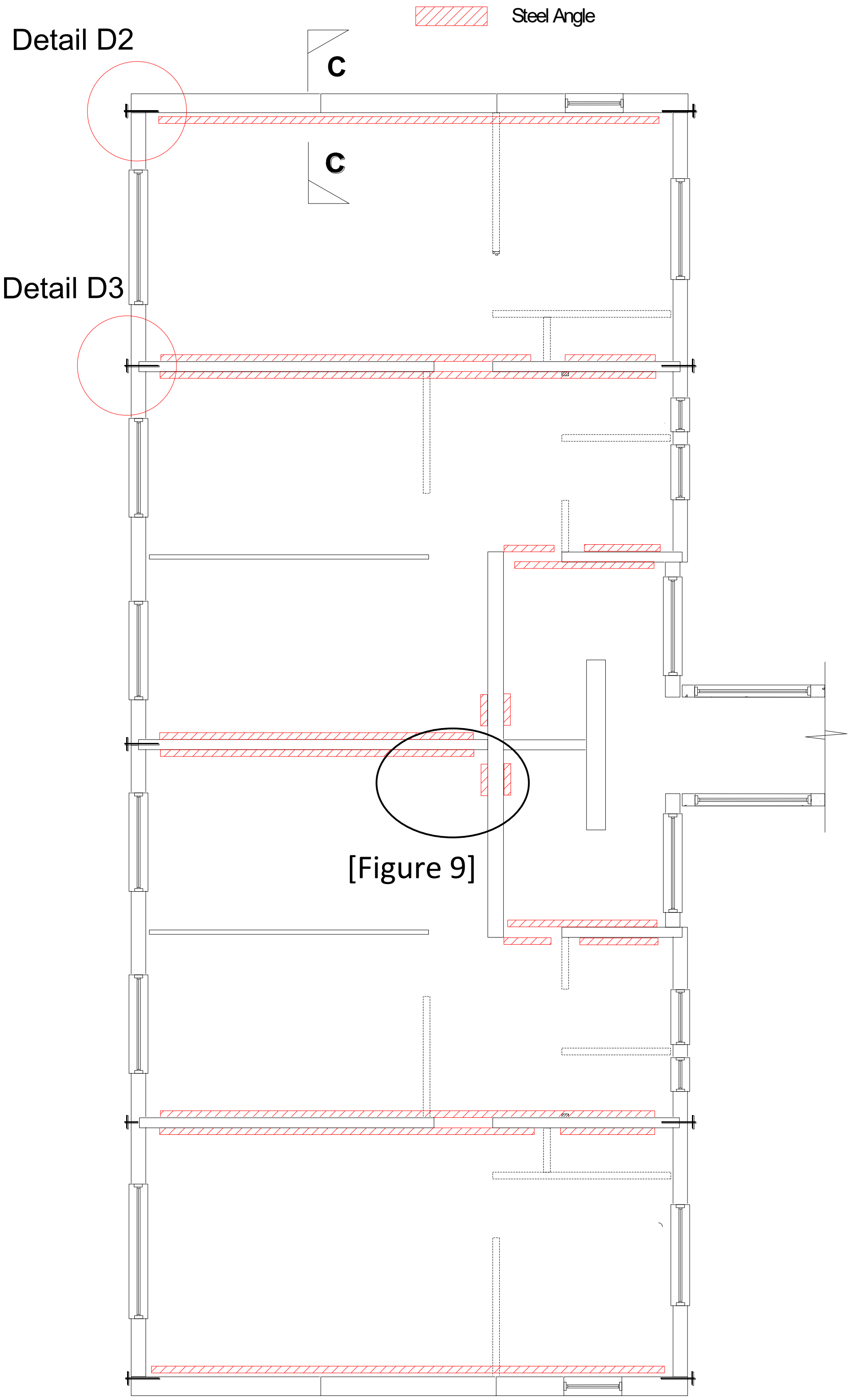
If access to more flats were available more investigative samples would have been obtained.

5.2 Existing Strengthening Works

Survey of the flats accessed revealed the existence some degree of strengthening works [Figures 9, 10, 11 & 12]. Steel angles were observed at ceiling level bolted into the ceiling and walls. It is assumed that the bolted steel angles constitute the strengthening works carried out following the Ronan Point gas explosion. The steel angles are installed on the load bearing walls at every level in the flats. Short length steel angles were also observed at the base of the flank wall panels only protruding out of the screed.

[Figure 9] Ceiling level Angles (See Figure 11 for plan view)

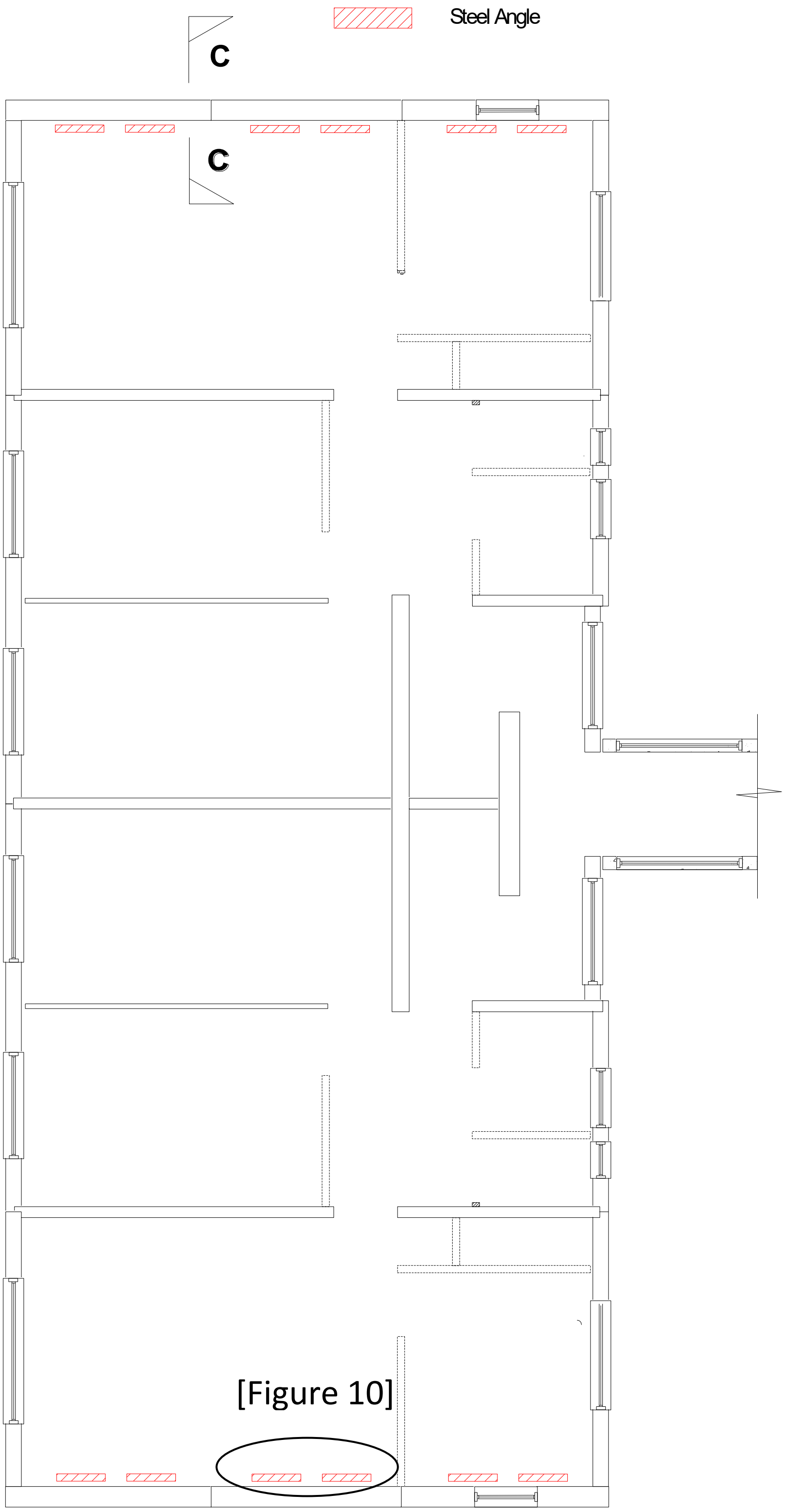
[Figure 10] Floor level Angles – Circled (See Figure 12 for plan view)



CEILING LEVEL VIEW

[Figure 11] Existing strengthening Angles

Angle sections - 100x100 RSA typical



FLOOR LEVEL VIEW

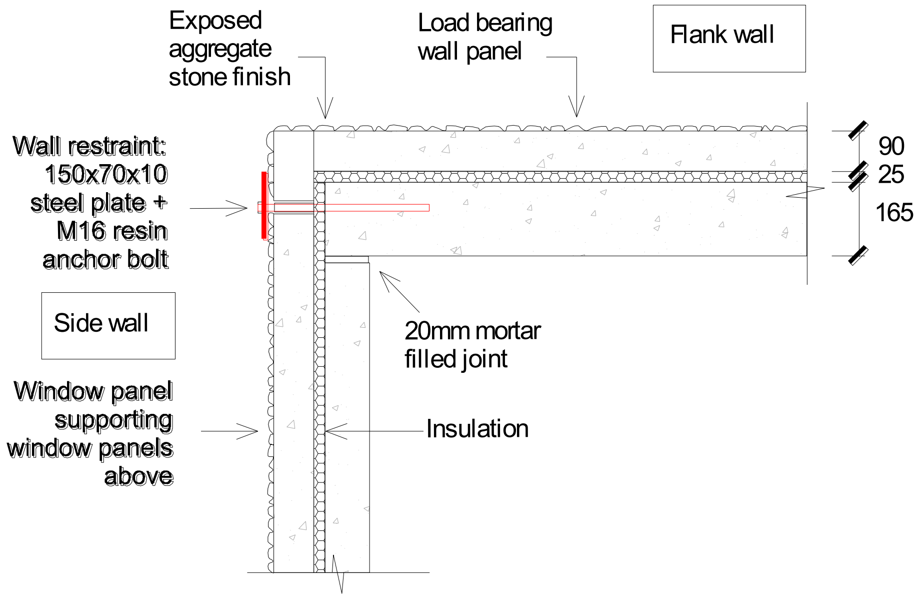
[Figure 12] Existing Strengthening Angles

Angle sections - 100x100 RSA typical

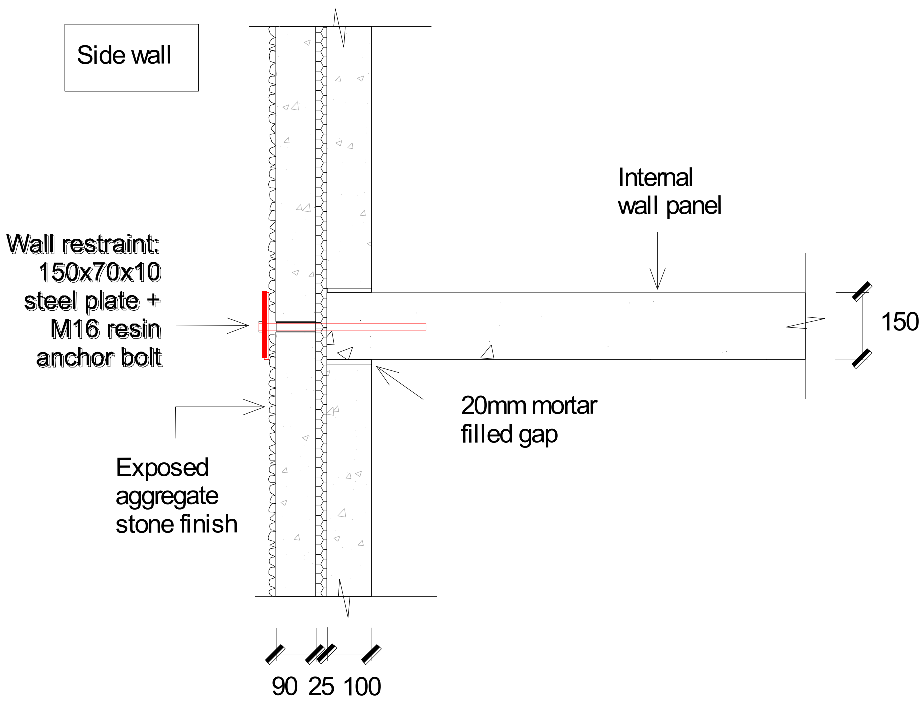
Wall restraints were observed on the buildings fixed to each and every level. These are shown in Figures 13, 14, 15 and 16 below.

[Figure 13] External wall restraint

[Figure 14] External wall restraints - Circled



[Figure 15] External wall restraints on the side wall corner of the high-rise.



[Figure 16] External wall restraints on the side wall of the high-rise.

5.3 Intrusive Survey Findings

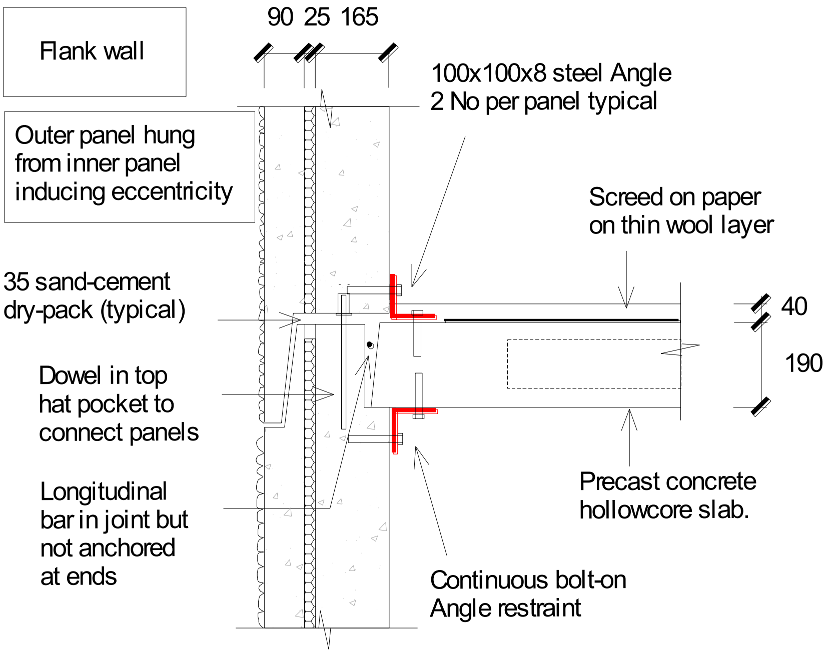
Structural Connections [Table 2]

	Connection	Observation
1	Flank wall-to-slab connection	Angle brackets and tie-back steel straps are used to tie the floors to the flank walls. See Figure 17. No observed reinforcement tying the flank walls to the concrete floors. The slabs were seated about 70 mm on the inner skin of the sandwich panel and is secured to the wall by a continuous 100x100x8 mm angle with 4M16 bolts per 2.5m wide panels. This bolted connection is seen to further secure the external flank walls to the floor slabs. The feet of the walls were dowelled and anchored to the slabs with 2 No 1.2m long steel angles per panel. The 12mm diameter bar in the floor joint placed longitudinally was not anchored at the ends. Although the flank wall appears better tied than the others, it is still inadequate for tie-back under collapse conditions.
2	Flank wall to flank wall connection	The inner skin of the vertical joints was stitched together with 8 mm projection loops and a 12 mm lacer bar. The lacer bar was not continuous into the upper or lower level panels. See Figure 21.
3	Cross wall to slab connection	150mm thick panel wall [type 2] with no clear ties to concrete slab or walls above and below. The top of the cross walls are bolted to the soffit of the concrete slabs with continuous steel angles similar to that of the flank wall but on both sides of the wall. The bases of the cross walls are not connected vertically or horizontally. See Figure 19.
4	Side wall to flank wall connection	The panels are recessed at the ends and only grouted. No mechanical connections were found. See Figure 15.
5	Side wall to cross wall connection	The side wall panel joints are grouted only. No mechanical ties observed. Stainless steel externally mounted tie-back plates were used as lateral restraint to stabilise the side wall panels. Two tie-back anchor plates were observed per panel. See Figure 16.
6	Side wall to slab connection	The only tie-back strap connection between side wall and slab was located at ground floor. The absence of these in the upper floors has resulted in the side wall panels bowing and gaps opening up especially between the double-panel side walls and the floor slab. See Figure 18.
7	Side wall to wall vertical connection	The side wall panels are dowelled vertically. Dowels from the panels below project up into a steel top-hat cast into the bottom of the panels above. See Figure 18.
8	Side wall to 65mm thick partition wall connection	The joint is grouted only. The partition panel is supported by the floor slab and dry packed top and bottom. No dowels were observed between the wall and slab.
9	Shear wall to Party wall connection	This was a grouted connection; no mechanical ties were found.
10	Shear wall to slab connection	Bolted steel angles at ceiling level. Head of wall restrained to side of slab with Angles.
11	The shear wall to wall vertical connection	2 No 32mm diameter anchor bars were observed at the ends of the shear walls with threaded couplers connecting them together vertically.

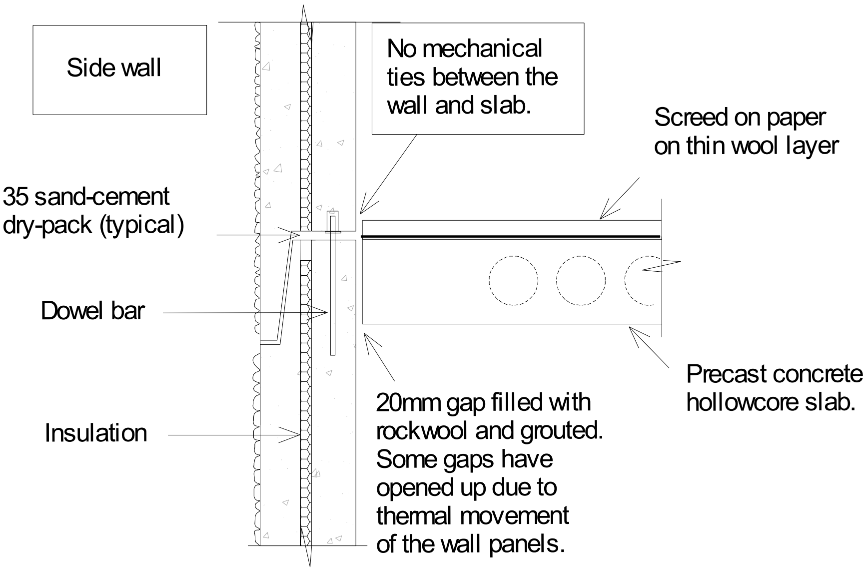
12	Stair to landing Connection	A halving joint connection was identified; however, no mechanical ties were observed between the stair flight and the landing slab. See Figures 28 & 39.
13	Landing to wall connection	The landing slabs were seated on concrete corbels at every floor level. No mechanical ties were observed. See Figure 29.
14	Link bridge to residential block slab connection	The link bridge slab is supported on beams both at the stair and residential block ends. The stair end connection is not tied, possibly considered as an expansion joint in design. Thermal movement of up to 10mm was observed in the floor joints between the access tower and residential block at the top level. See Figure 23.
15	Link bridge wall to slab connection	The link bridge wall panels are supported at each level on the slab edge. The wall panels are tied back to the bridge slab with two tie-back anchor plates per panel. See Figure 7.
16	Link bridge wall to wall connection	Grouted vertical joints only. No mechanical connections between walls.

5.4 Connection Photos and Details

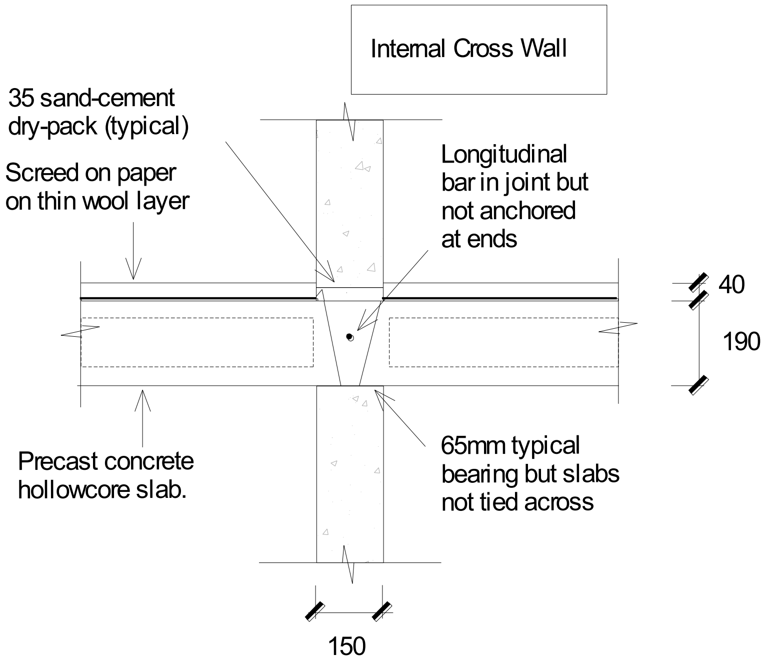
The intrusive investigation confirmed no inbuilt vertical and horizontal steel ties in the wall-to-floor and wall-to-wall connections, the requirement of which is stipulated by the Building Regulations for the robustness compliance of class 2B buildings. The diagrams below [Figures 17, 18, 19 and 20] show the surveyed existing joint construction.



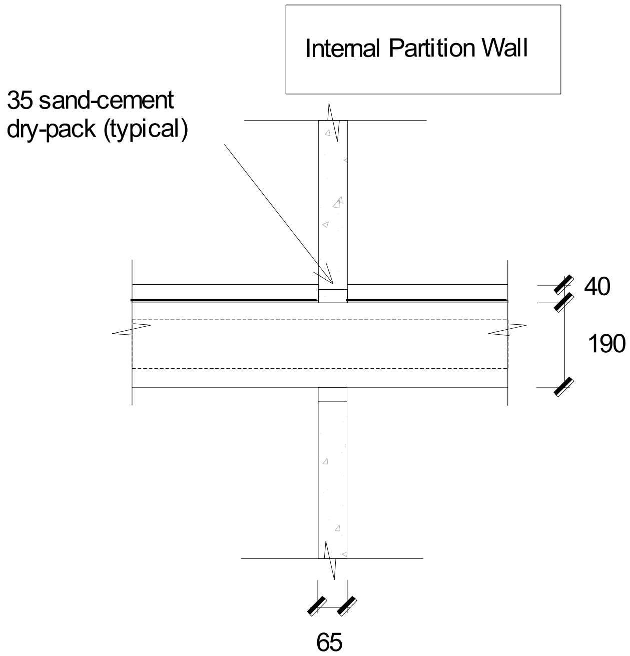
[Figure 17] Flank wall connection to the floor slab.



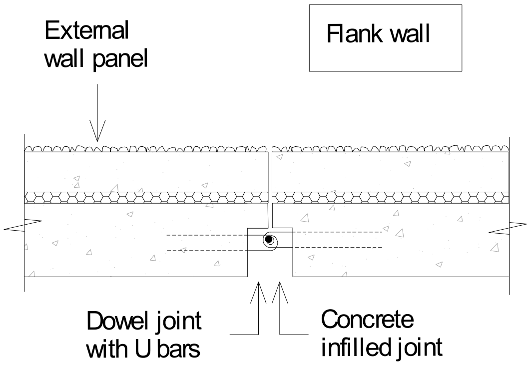
[Figure 18] Side wall connection to the floor slab.



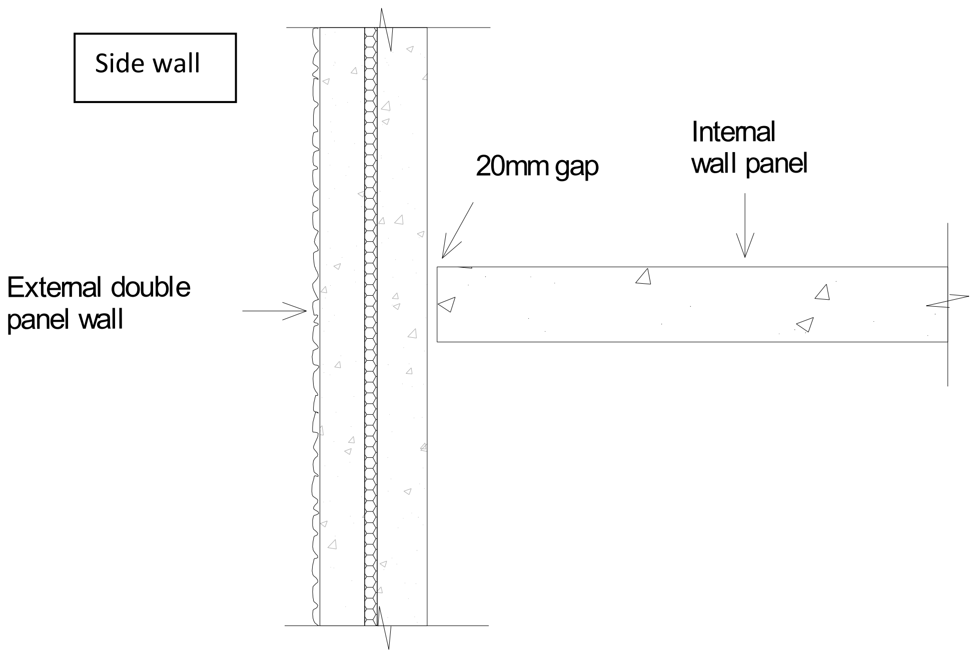
[Figure 19] Internal load bearing wall connection to the floor slab.



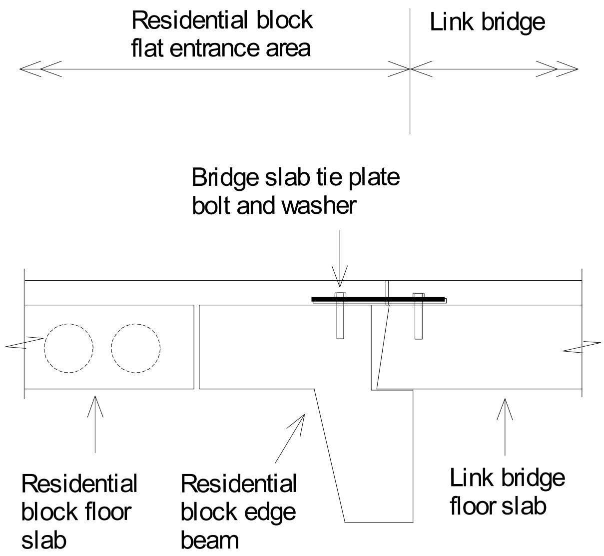
[Figure 20] Internal load bearing wall connection to the floor slab.



[Figure 21]: Flank wall panel connection

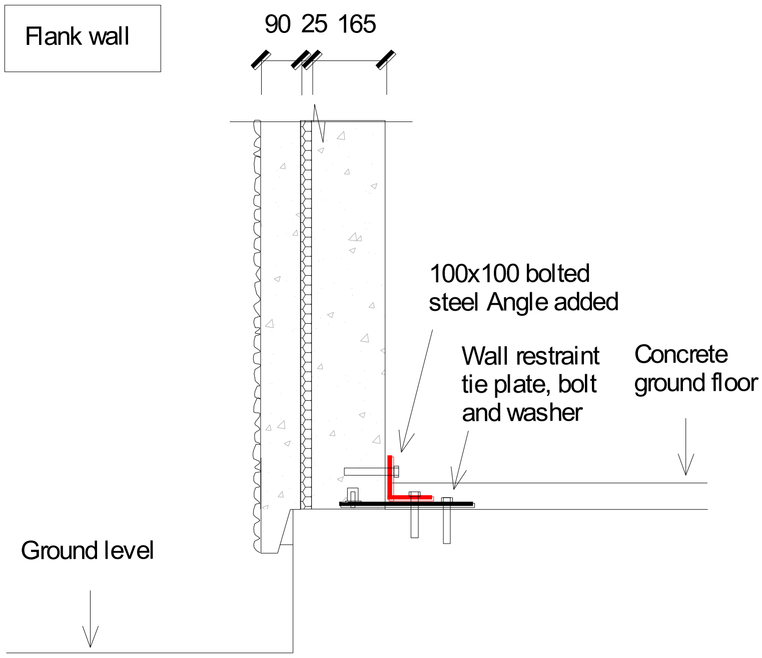


[Figure 22]: Internal non-load bearing wall connection



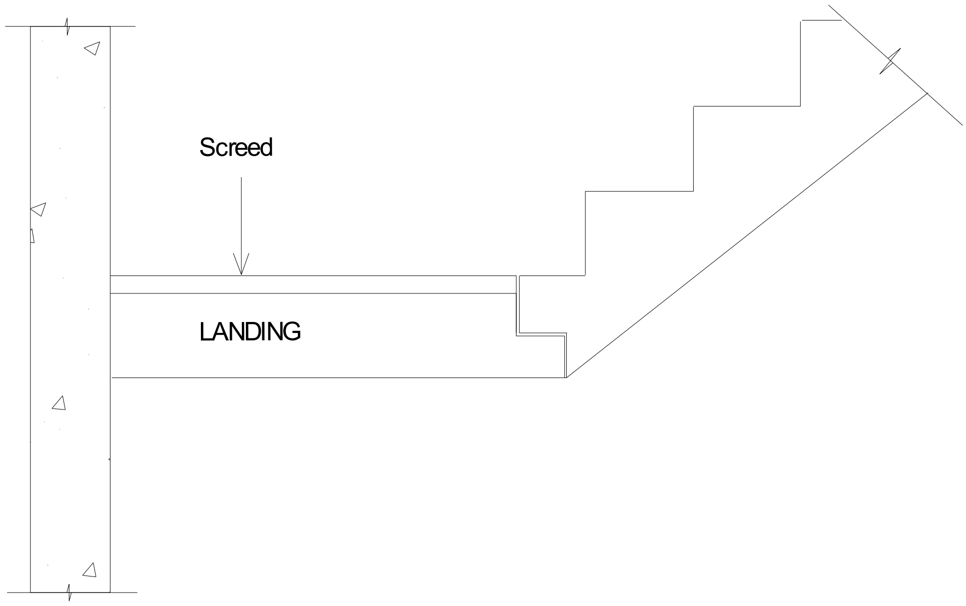
[Figure 23]: Link bridge slab bearing on edge beam

[Figure 24] Two tie-plates in Link Bridge slab tying it to the residential block

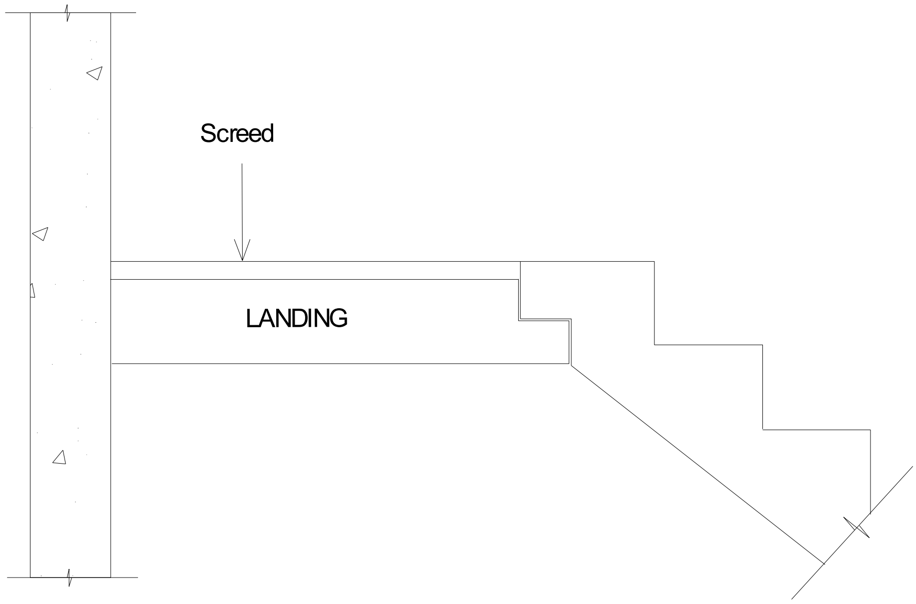


[Figure 25]: Ground floor flat wall panel connection

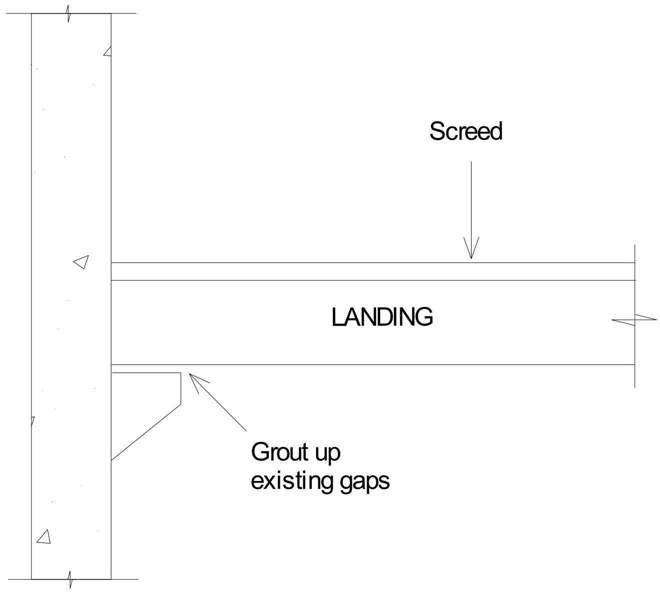
[Figure 26]: Ground floor flat wall-restraint tie plate and bolt photo



[Figure 27]: Stair and landing details



[Figure 28]: Stair and landing details



[Figure 29]: Stair landing support detail

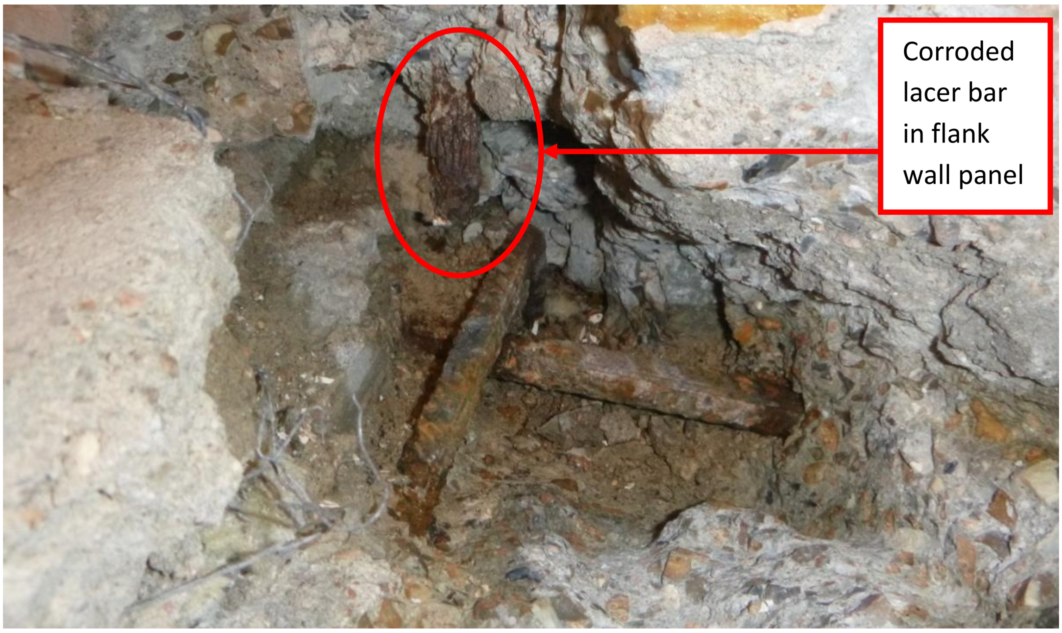
[Figure 30]: Stair landing support photo

[Figure 31] Tension bars in ends of 235mm thick shear wall

[Figure 32] Bolted Angle fixing into flank wall

5.5 Joint reinforcement corrosion and water ingress

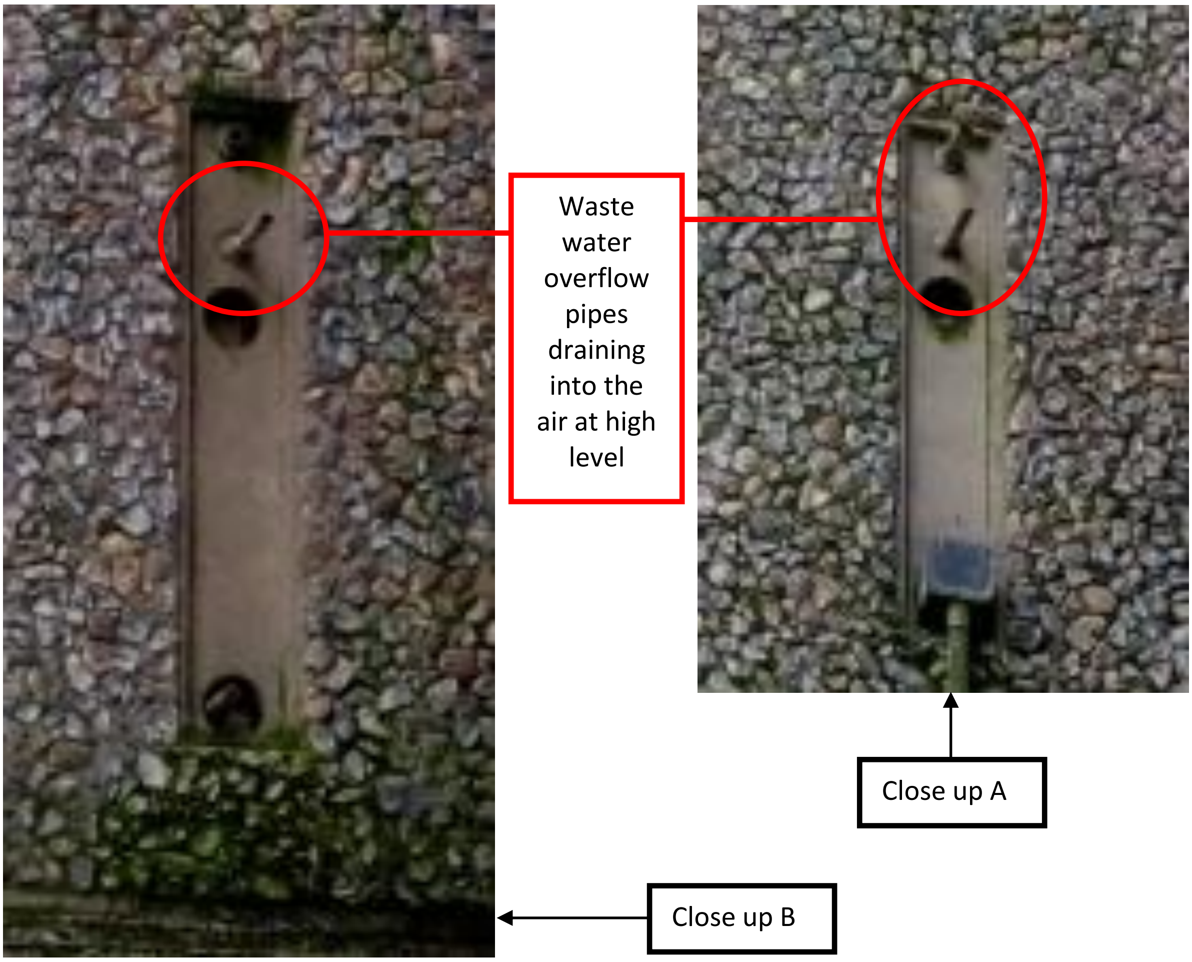
Some degree of reinforcement corrosion was observed in the flank wall of the ground floor flat.



[Figure 33] Reinforcement corrosion found in flank wall panel

Long term continual water flow onto the rear walls of the residential blocks will eventually encourage reinforcement corrosion when damp reaches into the fabric of the concrete panels.

[Figure 34] Waste water overflow pipes (circled) sticking out of the rear wall of the block. White residue water stains were observed on the dry face of the rear walls, and green coloration of vegetation on the damp face.



[Figure 35] Close-up on overflow pipes to Figure 19

The Stage (I) report recommendations addressed the issue of waste water spilling out onto the side of the buildings as follows:

(a) Inspect the kitchen and bathroom waste water drain pipes (including the sink) in all the flats for leaks and faulty connections. Any water leaks found are to be stopped and the joints repaired and/or sealed.
(b) Inspect the kitchen and toilet cistern overflow to all flats for faulty mechanism. The overflow pipes which drain out into the air are to be redirected into the internal drainage or connected externally to a new downpipe which will take the overflow water into the external underground drain.
(c) The mastic seals to the concrete panel joints are to be surveyed and faulty seals are to be remedied to stop rain water ingress into the property. Driving rain into the link bridge also requires attention.

5.6 Concrete durability

Carbonation and chloride levels in the concrete were tested in a few locations. The levels measured were generally low. Testing for chlorides gave negligible results.

[Figure 36] Carbonation identification coloration in soffit of concrete floor

5.7 Overall building movement

High-rises are expected to move to some degree under wind loading. Survey of the buildings uncovered no issues of concern or distresses that may be attributed to overall building sway movement.

5.8 Thermal effects

The component panels in such high-rise buildings are expected to move to some degree under thermal effects. In most of the flats surveyed in both Hartopp and Lannoy Point cracks were observed in the finishes all-round the external wall panels. The consistency of this observation in most of the flats surveyed suggests that movement is cyclical but not of structural concern at the time of the survey. These movements were mainly observed in the top two levels of the link bridges.

5.9 Gas supply and heating system

The type of heating system in the tower blocks was electric wall-mounted heaters. These were observed in all the flats accessed. All previously installed gas pipes have been removed from the tower blocks following the Ronan Point explosion. For this reason, the accidental blast loading used in the assessment of the tower blocks is 17kN/m2 as suggested in BR511 titled '*Handbook for the structural assessment of large panel systems (LPS) dwelling blocks for accidental loading*'. No previous reports were found pertaining to the high-rises that reported distresses or structural damage under normal loading, or the occurrence of a gas explosion in the past.

5.10 Fire risk and related damage

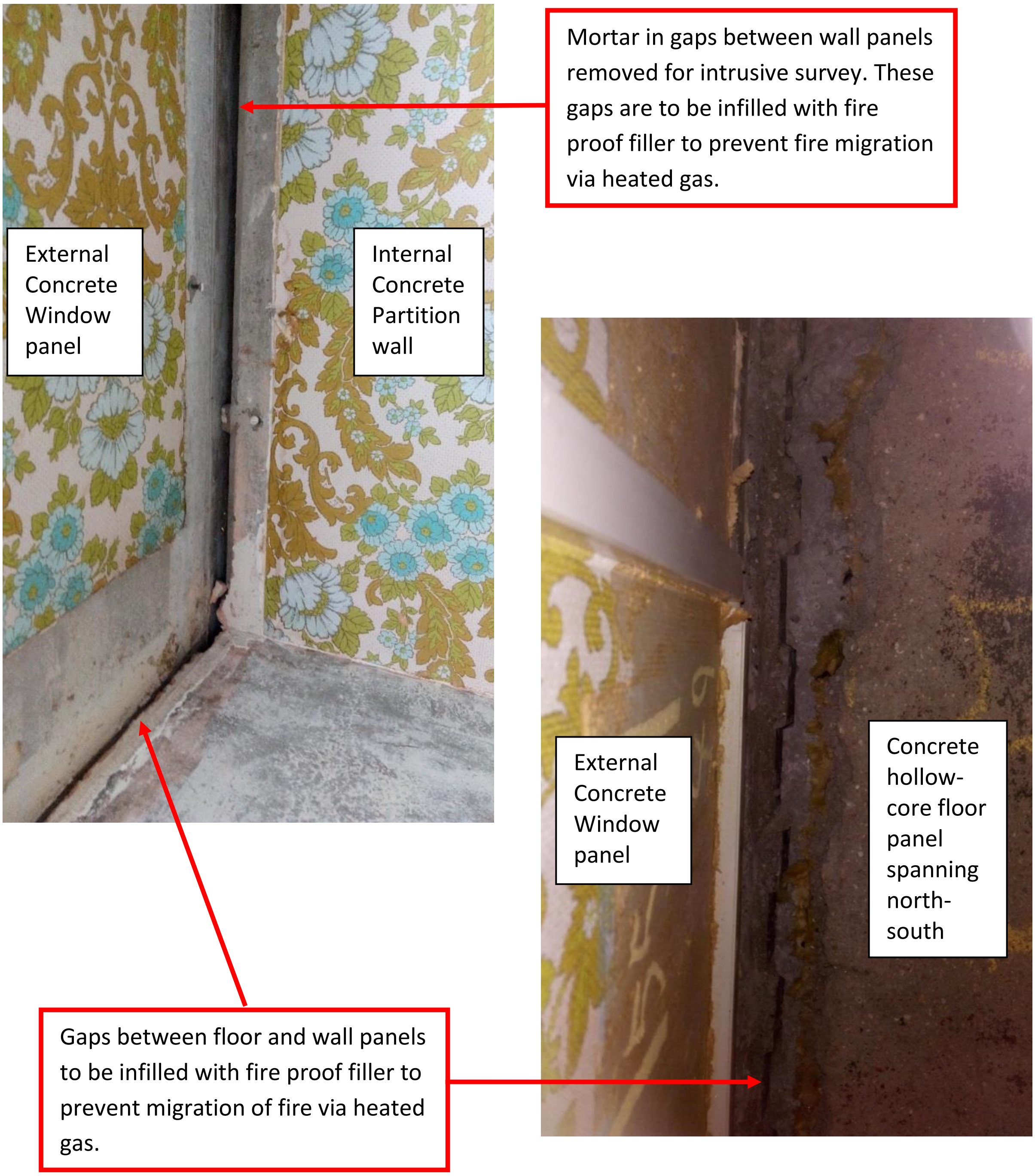
The failure of the concrete slab under fire conditions usually occurs in the form of spalling which is the progressive deterioration of the surface exposed to heat. No signs of fire related damage were observed in both high-rise blocks except in the communal airing room on the 12th floor at Hartopp Point where minor localised fire damage was observed in the wall panels.

The wall panels and floor construction comply with the fire resistance requirement of the London Constructional Bylaws and London Building Acts of the time.

5.11 Concrete Strength

With regards to the strength of the concrete walls and slab, investigation into the material strength was carried out by Martech Limited. The Schmidt hammer readings ranged from 44 to 76.

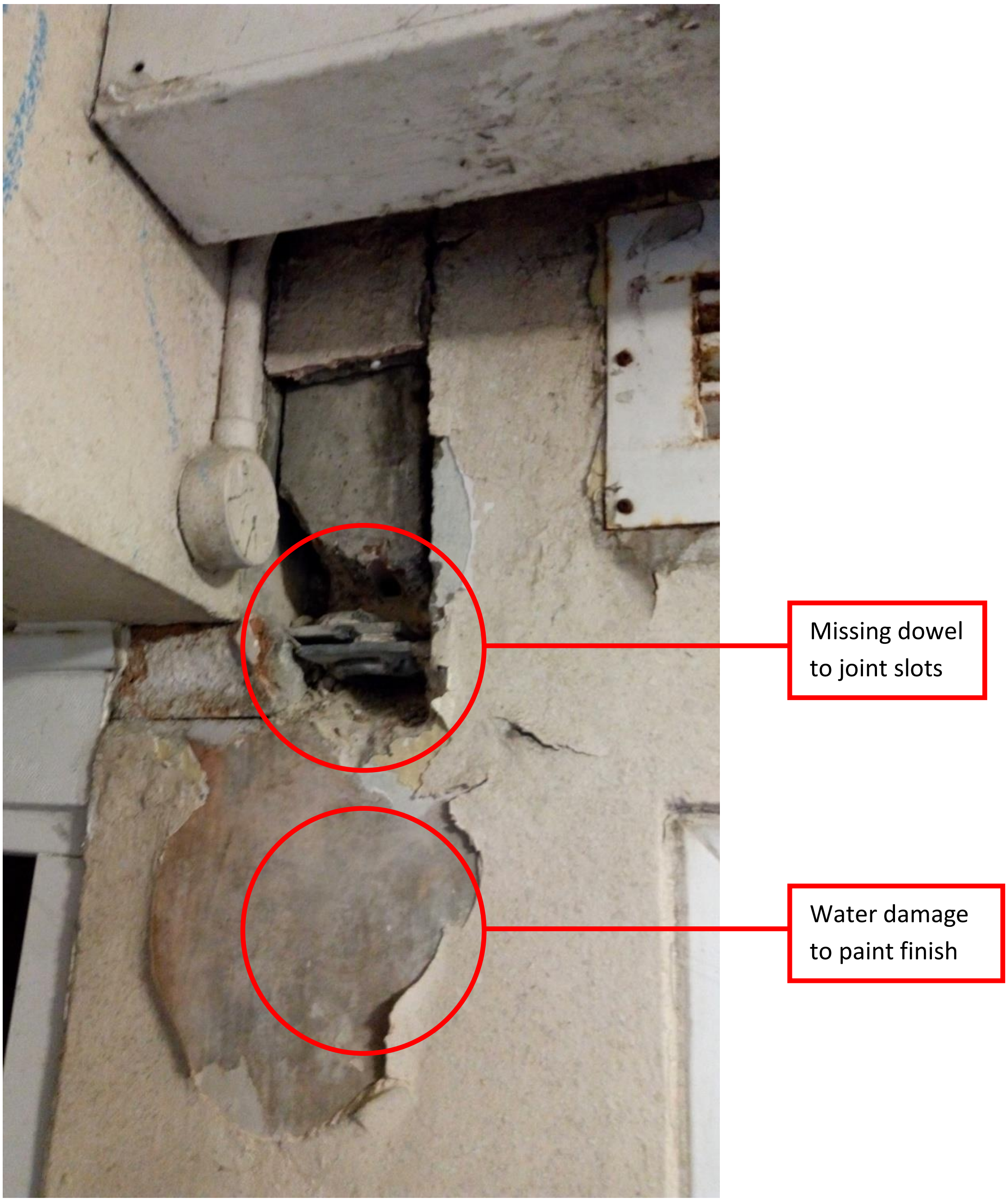
5.12 Gaps in the panel joints



[Figure 37] Gaps in panel joints

5.13 Construction defects

Construction defects were found in the intrusive investigation of the access core stairway and link bridge. Photos of the observations are shown below.



[Figure 38] Dowel missing in link bridge wall joint

The joint face in the top corner of the link bridge was broken out to expose the jointing and to investigate why there were cracks around the jointing. Some water damage to the paint finish was observed.

The stair landing at level 12 was investigated to see whether the precast units were tied together by reinforcement. Figure 15 below is a photo of what was uncovered. No tie back reinforcement was found in the connection.

[Figure 39] Tie-back reinforcement missing in landing-to-stairs connection.

No tie-back reinforcement was observed in the breakout. It is therefore assumed that the stairs and landing construction at all levels were built or connected together without internal ties.

6.0 Assessment Stage 3 - Assessment of Block under Normal Loading

The service period of Hartopp and Lannoy high-rise blocks is now in excess of 45 years. Structurally the external fabric of the buildings appears to be in reasonable state of repair. No visible distresses were observed which can be attributed to foundation settlement. A number of concrete infilled joints observed, especially in the link bridges, were cracked due to thermal and/or building sway movement.

The central stair and lift tower block appeared sturdy. No signs of distresses were observed. No cracks were observed in the concrete stairs up to the top floor in both buildings. It was noted that the stairs and landing slabs were not tied-in at the bearings.

Outside of the residential towers footprint and in the maintenance office by the basement level garages, some concrete spalling due to reinforcement rust was observed in two of the overhead concrete beams. Spalled concrete and exposed rusted reinforcement were also observed in the overhead concrete beams in some places along the garage exit driveway.

Considering the age and the visual observations made of the two residential tower blocks, the buildings continue to perform their required function carrying the normal design dead and live loads including wind without incident.

Load assessment showed that the current standard design Dead, Live and Wind load stresses are within allowable.

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7.0 Assessment Stage 4 - Assessment of Block under Accidental Loading

APPROVED DOCUMENT A: A3 REQUIREMENT

Requirement for Class 2B buildings
*Provide effective horizontal ties, as described in the Codes and Standards listed under paragraph **5.2** for framed and load-bearing wall construction (the latter being defined in paragraph **5.3**), together with effective vertical ties, as defined in the Codes and Standards listed under paragraph **5.2**, in all supporting columns and walls.*

*Alternatively, check that upon the notional removal of each supporting column and each beam supporting one or more columns, **or any nominal length of load-bearing wall** (one at a time in each storey of the building), the building remains stable and that the area of floor at any storey at risk of collapse does not exceed 15% of the floor area of that storey or 70m2, whichever is smaller, and does not extend further than the immediate adjacent storeys (see Diagram 24). Where the notional removal of such columns and lengths of walls would result in an extent of damage in excess of the above limit, then such elements should be designed as a 'key element' as defined in paragraph **5.3**.*

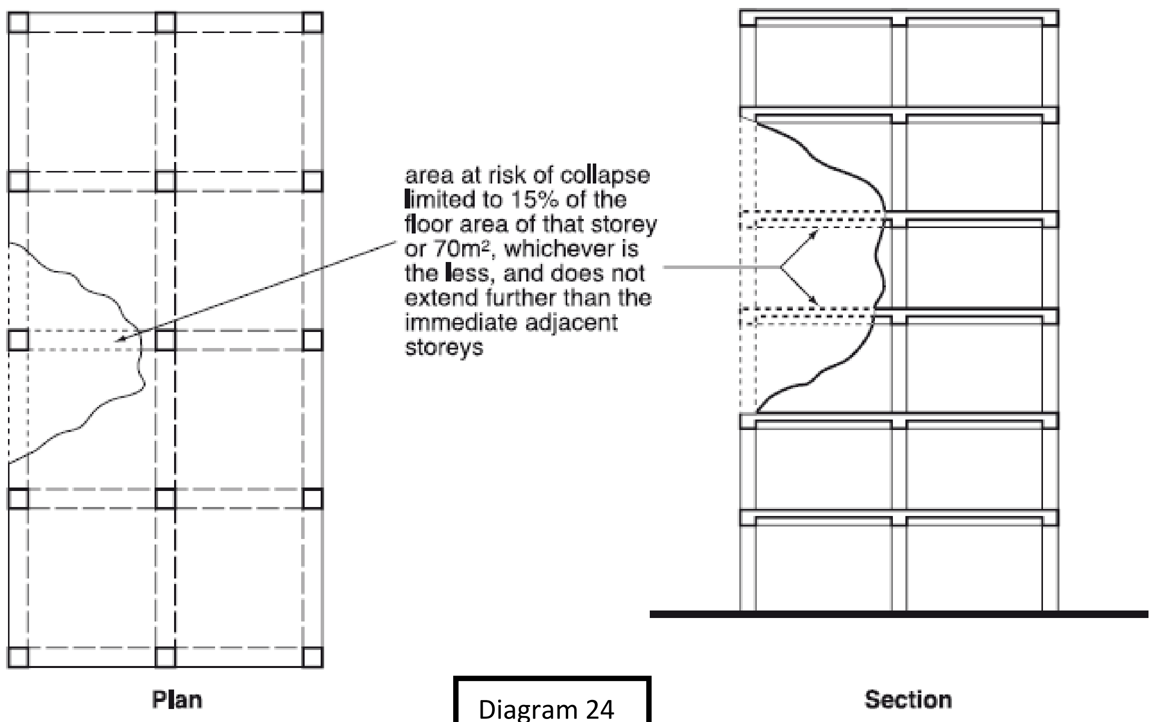


Diagram 24

Compliance requirement in summary

1. Identify the horizontal and vertical ties else provide for compliance.
2. Identify key elements.
3. Show that the removal of a wall or column will cause only limited damage.

<u>1 - Horizontal & Vertical Ties</u>
The intrusive survey carried out in Flat 1 (Lannoy), Flat 27 (Lannoy) and Flat 31 (Hartopp) identified no inbuilt horizontal and vertical ties in the panels. Based on this information it is presumed that the two tower blocks were designed without horizontal and vertical ties therefore making the construction non-compliant with the requirements for Class 2B buildings as stipulated in the Approved Document A to the Building Regulations.

<u>2 - Key Elements</u>
No key elements were identified in the construction of the tower blocks.

<u>3 – Damage Assessment/ Alternative Load path</u>
Further to the intrusive survey and the creation of the plans, section and connection details, the robustness test was performed removing one load bearing wall at a time as prescribed in the Approved Document A to the Building Regulations. The collapse mechanisms were determined, and the conclusion was that the A3 test failed for both tower blocks.

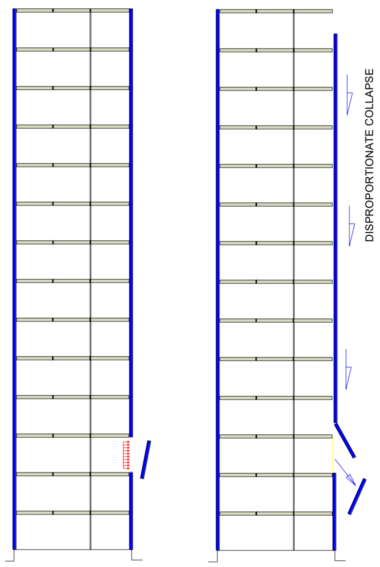
Load check on the doubly reinforced flank wall under accidental blast loading of 17kN/m2 showed that the flank wall has insufficient strength in itself in bending to resist the blast. Likewise, the floor slab proved insufficient in strength to take the reduced blast loading.

A3 Assessment Summary [Table 3]

Approved Document A: A3 Damage Test						
Test Assumptions 1 – No effective vertical and/or horizontal ties are built into the structure. 2 – No key elements are built into the structure. 3 – The two blocks are a stack of precast floor and wall panels assumed simply supported. 4 – Storey height ‘H’ is approximately 2.6m 5 – Max length of wall to be removed = 2.25*H						
Element	**Extent**	**Function**	**A3 Test Action**	**Structural Response**	**A3 Compliance**	**Remedy**
Flank wall [Figure 41]	Double panel	Support to floors and walls above	Remove [Figure 41a]	Walls and floors above will collapse [Figure 41b]	Fail	Strengthen structure
Cross wall [Figure 42]	Single panel	Support to floors and walls above	Remove [Figure 42a]	Walls and floors above will collapse [Figure 42b]	Fail	Strengthen structure
Side wall [Figure 40]	Single panel	Support to walls above only	Remove [Figure 40a]	Walls above will collapse [Figure 40b]	Fail	Secure walls

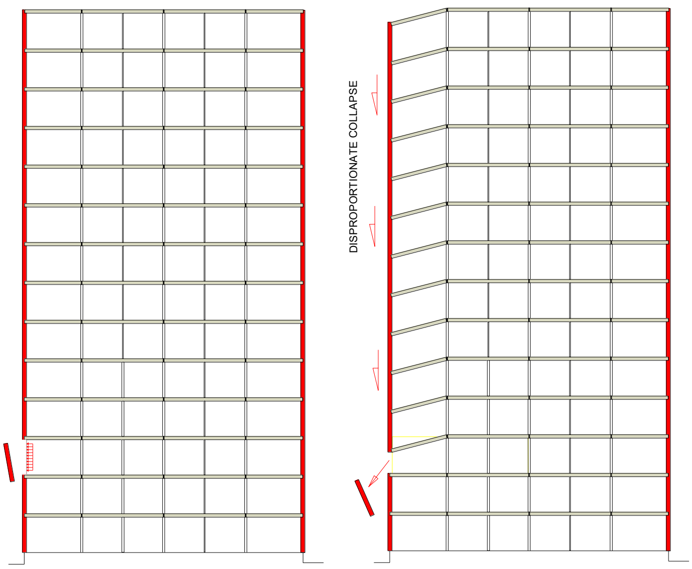
PROGRESSIVE COLLAPSE SCENARIOS

Diagramatic representation of Table 3: Removal of a wall panels resulting in disproportionate collapse.



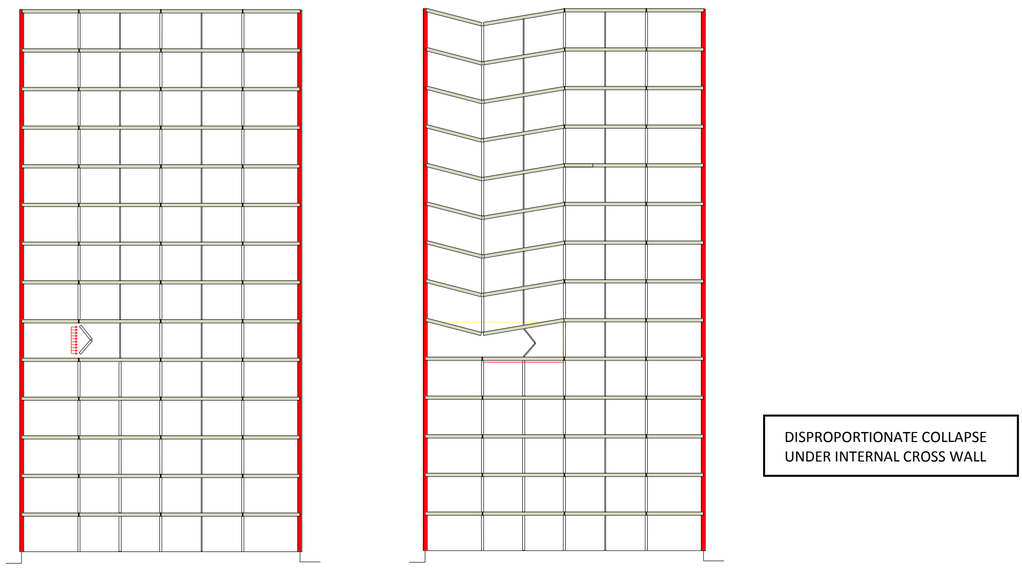
(a) (b)

[Figure 40] Short section view
Side wall removed



(a) (b)

[Figure 41] Long section view
Flank wall removed



(a) (b)

[Figure 42] Long section view
Internal cross wall removed

8.0 Conclusion

Structurally, both Lannoy and Hartopp high-rise buildings appear to have performed satisfactorily over their service period of over 45 years. No signs of historic or newly occurring distress were observed under the service dead and live loads in terms of fractured wall and floor panels. Cracks in mortar filled joints were observed in places mainly due to thermal movement. No wall panels have been reported dislodged or moved out of position in the 45 years of service suggesting that wind loading on the buildings have been accommodated and resisted without signs of distress.

Based on the information obtained from the intrusive survey, it is concluded that the buildings were not designed to accommodate damage and prevent disproportionate collapse in the event of an accidental damage. The construction is therefore non-compliant with the requirements for Class 2B buildings as set out in Approved Document A (Structure) to the Building Regulations 2010 (as amended).

Structurally the external fabric of the building appears to be in a reasonable state of repair. Steel restraint plates were observed externally securing the wall panels to the front and back elevations of the residential blocks. Further to the Ronan Point disaster in 1968 strengthening works have been carried out to both tower blocks visible internally. Steel angles were observed internally secured at high level to the main load-bearing walls. Gas lines to the tower blocks have been removed and all cookers and central heating appliances have been changed to electric. Nevertheless, portable gas canisters brought into the premises is a real possibility to be considered in the management of the premises in the future.

Some signs of minor movement were observed in the link bridge at the 12th and 13th floor of Hartopp point. Similar observations were made at the 13th floor of Lannoy Point. These movement observations gave no cause for concern at the time of the survey.

9.0 Recommendations

To meet the robustness requirements of Approved Document A3 to the Building Regulations for class 2B buildings there is only one option available for the towers moving forward. This option is to strengthen the tower blocks to bring them up to ADA3 compliance. For an indicative solution see Appendix A.

10.0 List of References

1. Handbook for the structural assessment of Large Panel Systems (LPS) dwelling blocks for accidental loading. [BRE 511].

2. Approved Document A to the Building Regulations 2010.

3. Martech Technical Services Report 15th December 2017

4. Structural Robustness of Steel Framed buildings (SCI 2011)

5. Technical Notes by SCI with the ISE (Jan 1999)

6. How to Design Concrete Buildings to Satisfy Disproportionate Collapse Requirements. (The Concrete Centre)

7. The Design of Laterally Loaded Walls [J Morton]

8. Practical Guide to Structural Robustness and Disproportionate Collapse in Buildings [ISE 2010]

APPENDIX A: A3 INDICATIVE REMEDIAL WORKS SCHEME

REMEDIAL WORKS DESIGN PHILISOPHY

The design concept is based on the alternative test for 2B buildings which allows the removal of load bearing walls, one at a time, and considers the collapse mechanism. The consequences of each simulated damage then suggests the use of vertical and horizontal ties and any associated works as remedy.

Loading calculations have demonstrated that precompressions loads at lower levels of the highrises will enhance the resistance of the wall panels to lateral loading, and a blast load of 17kN/m2 may be resisted by panel precompression. However, considering the discovery of (a) defective construction which may affect the reserve of strength in the construction, (b) the floor slabs not being strong enough to take a blast loading of 17kN/m2, (c) the lack of sufficient slab bearing (<90mm), and (d) the low level of reinforcement observed in certain wall panels, it is our view that the use of precompression loading will not guarantee a safe remedial works design. Therefore, for the purpose of this design precompression loading and its effects are ignored.

Vertical ties in the form of steel rectangular hollow sections, and horizontal ties in the form of Angles & Flats are installed at all levels. The use of steel hollow sections as vertical ties and columns help to reduce concrete panel size removed and provide better fixings of the external wall panels to the internal cross walls. Steel Angles form the horizontal ties along the load bearing walls including the side and flank walls, while the flats form the horizontal ties supporting the floors. The Angles also provide secure attachment of the walls to the floors much more than is existing.

The vertical connections of the hollow section steelwork is via the concrete floors. The head and base plates to the hollow sections will have three or four bolts connecting the plates together through the floor slab providing tie continuity.

Horizontal ties will connect together via plates some of which will require detailed design but are presented simply for this scheme. Most connections are bolted but some may require welding on site. All horizontal ties are installed at ceiling level and fixed to the floor slab.

Under explosion conditions it is assumed that the floor slab will fail upwardly but rest down onto the steel ties damaged. The vertical ties are of sufficient strength to resist the limited blast loading and remain in place albeit with the attached wall panels probably blown out. The remaining adjacent wall panels and hollow section posts will take the compression loads under damage conditions whilst the rest of the system will work in shear and tension to prevent any disproportionate collapse.

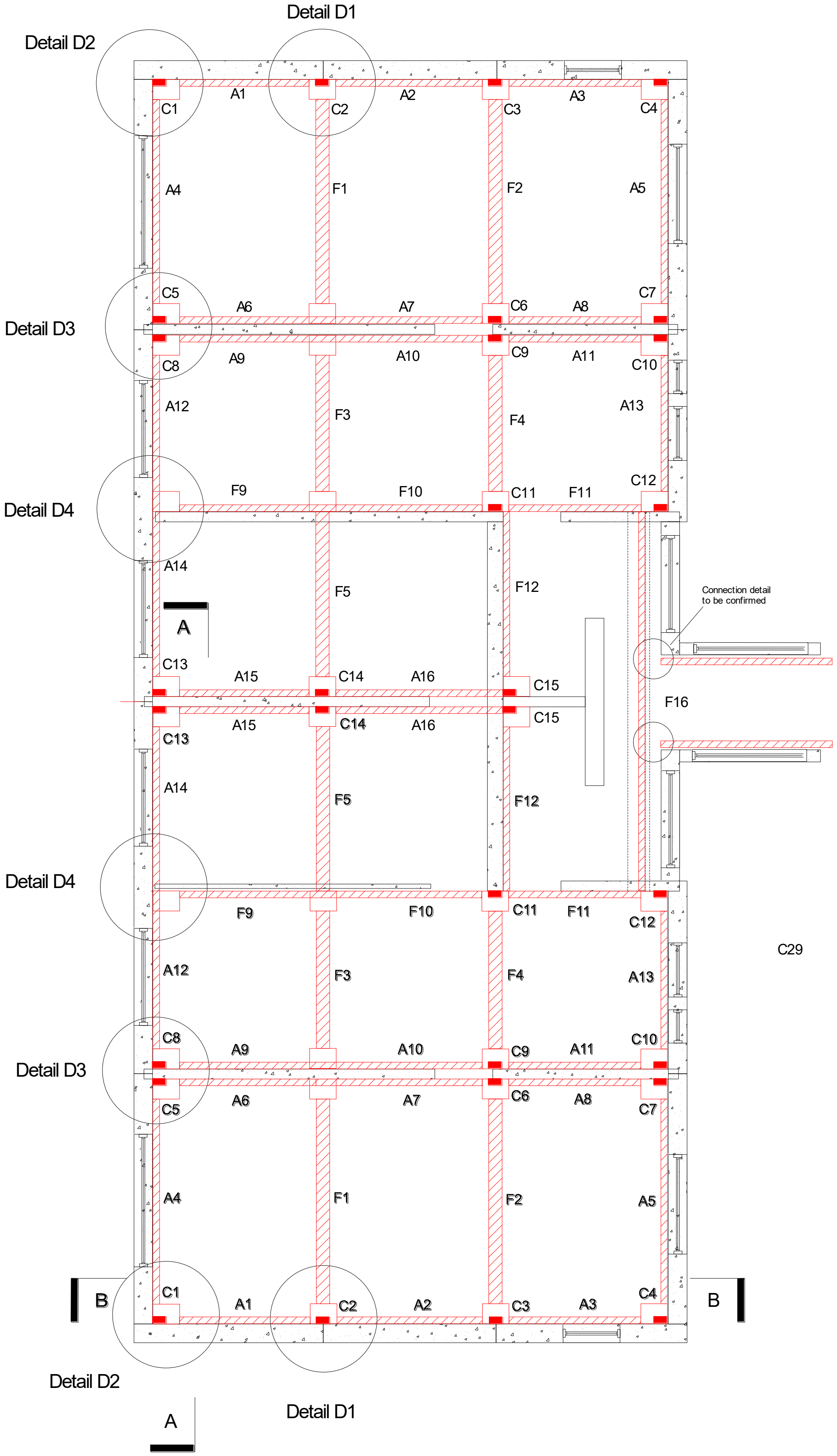
The lift and stair core will also be provided with vertical and horizontal ties throughout. The scheme design is shown on the drawings.

Information for costing purposes

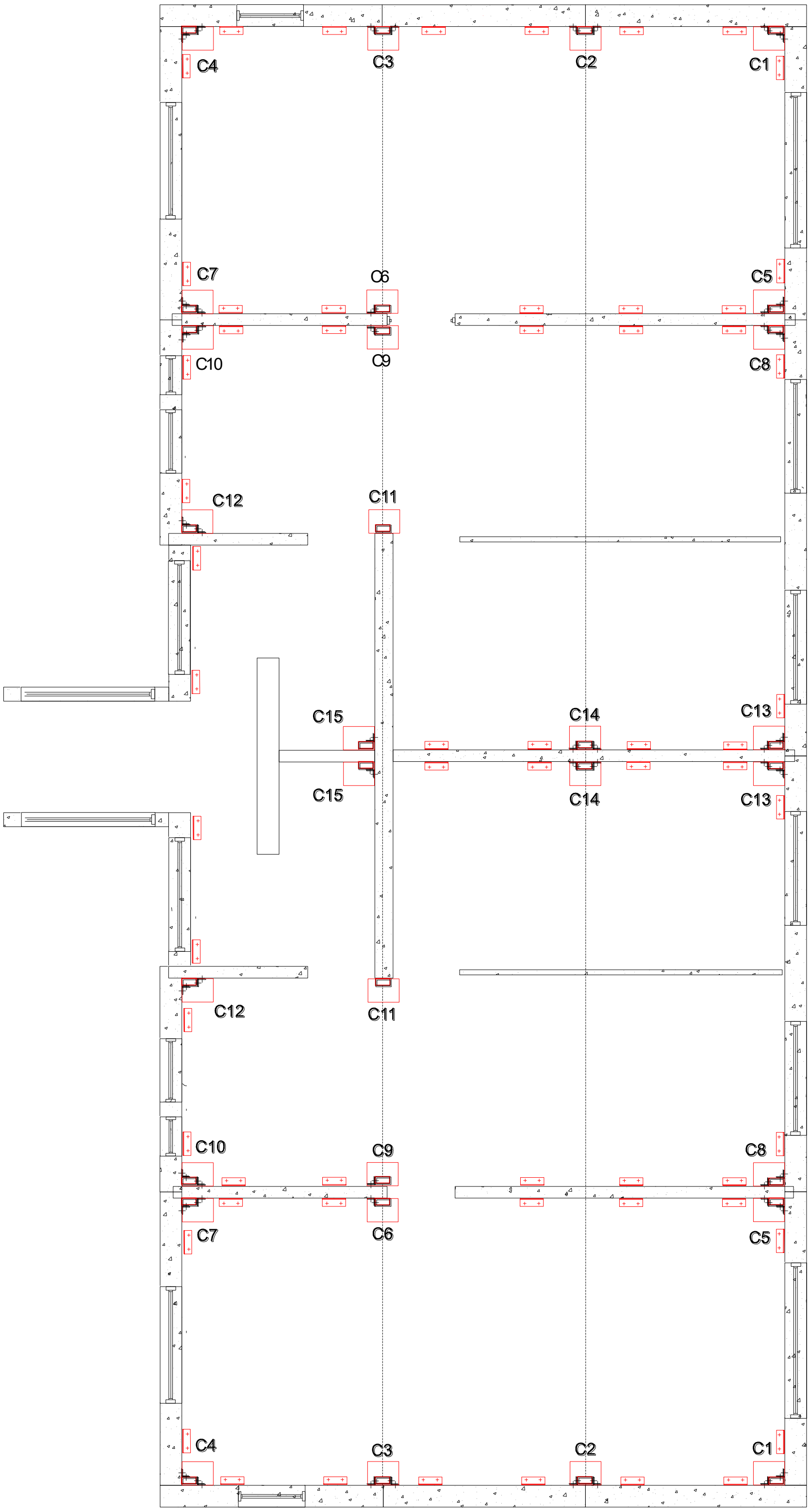
With regards to the costing of the project the scheme design will provide a reasonable estimate of works costs. Cost considerations will include:

1. Steel fabrication and supply.
2. Steelwork erection and concrete breakout works including site cutting, drilling, bolting, welding etc.
3. Asbestos removal.
4. Decanting of residents including tenants and lease holders of at least an entire wing.
5. Prevention of fire spread by sealing all gaps with fire resisting fillers.
6. Structural fire resistance by covering all steelwork with plasterboard.

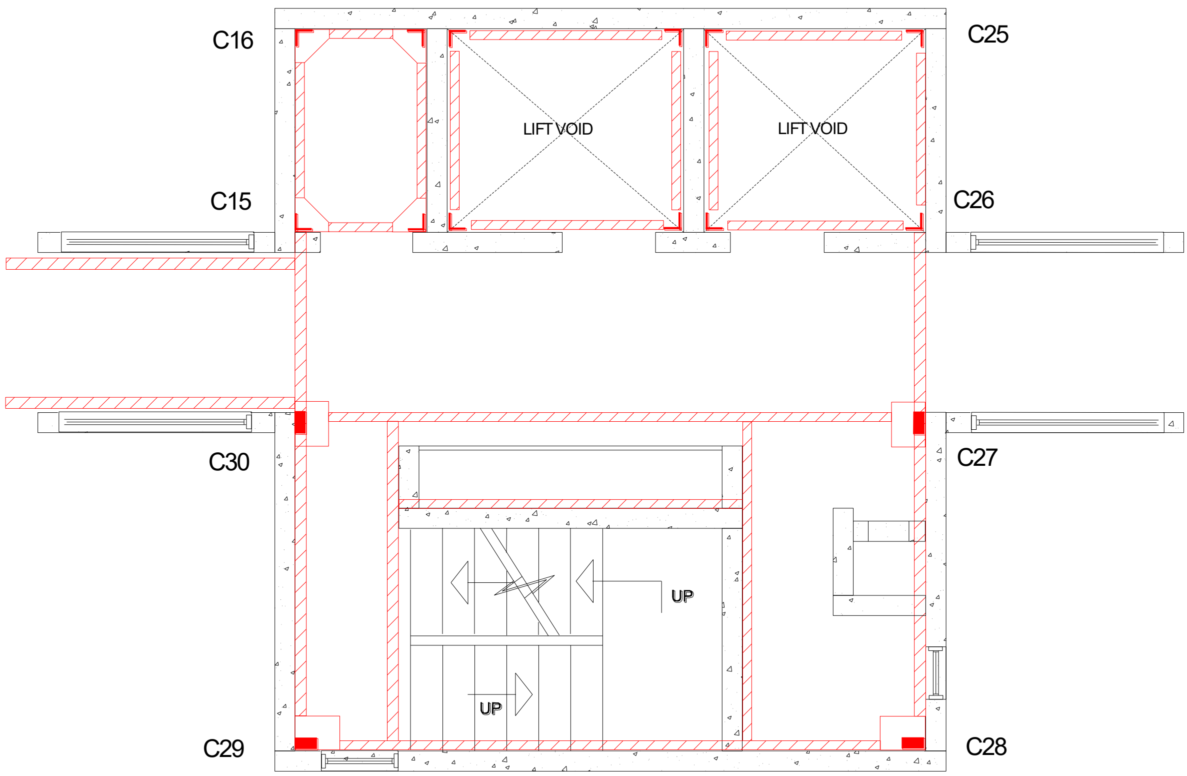
7. Installation of smoke and/or fire alarms as per fire specialist recommendations.
8. Installation of sprinkler system as per specialist recommendations.
9. Installation of dry lining insulation board to the external walls to all flats for better heat insulation. Some appartments have it while others do not.
10. Installation of external waste water down pipe from top flat kitchen down to ground.



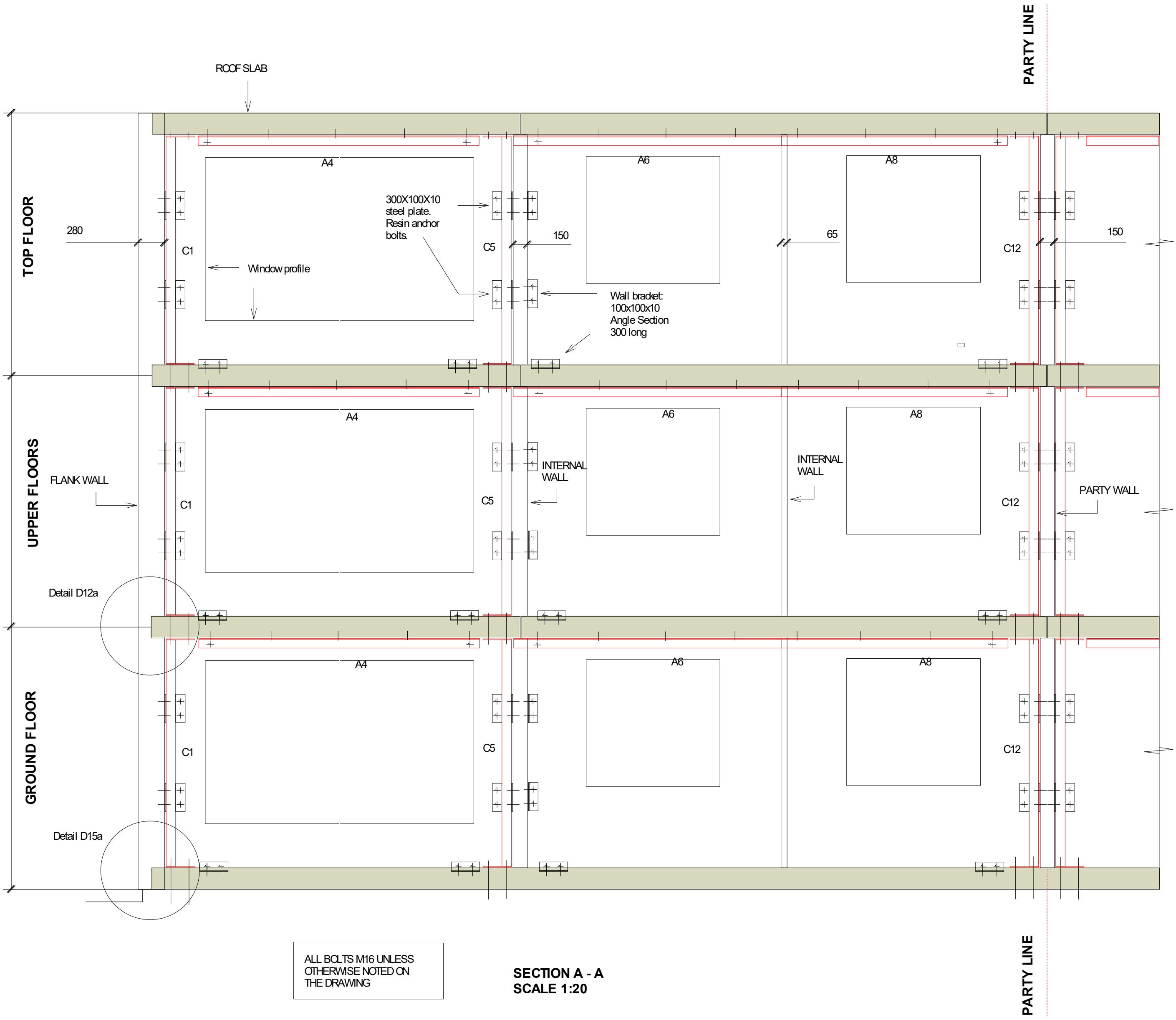
[Figure A1]: Indicative remedial works steelwork plan – ceiling level view
Columns C1 – C15: 200x100x10 RHS typical
Angles A1 – A16: 100x100x10 RSA typical, Flats F1- F5: 200x10 typical, F12: 100x10.



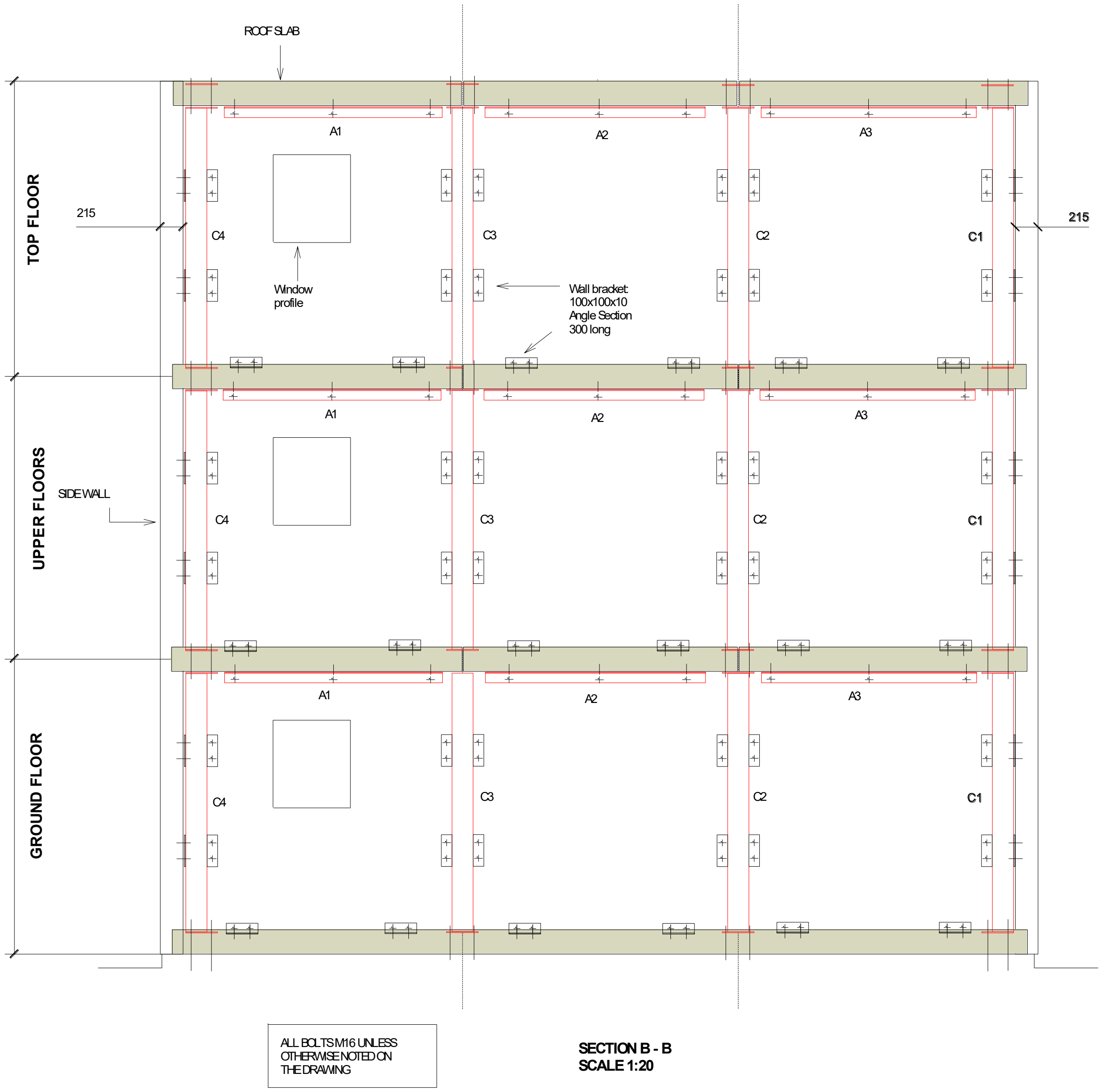
[Figure A2]: Indicative remedial works steelwork plan – floor level view
Columns C1 – C15: 200x100x10 RHS typical



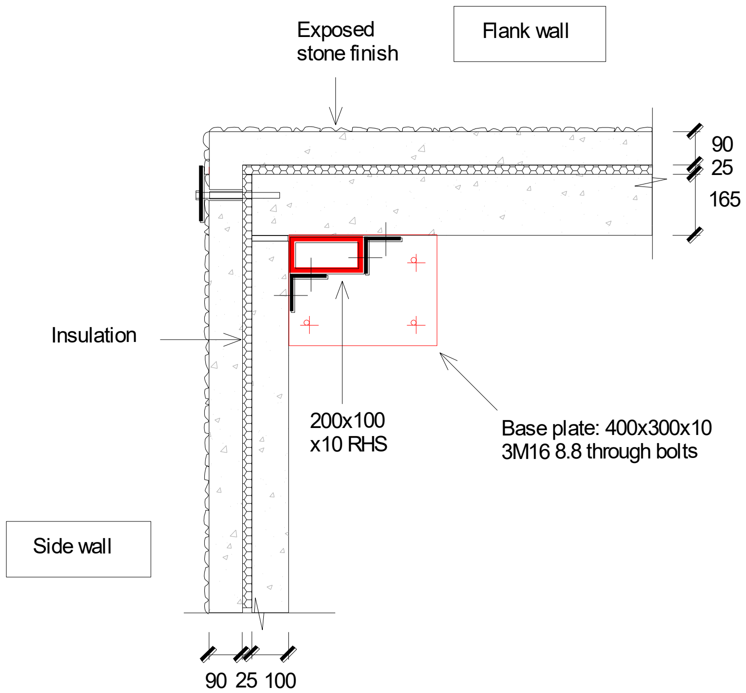
[Figure A3]: Access core indicative steelwork plan – ceiling level view



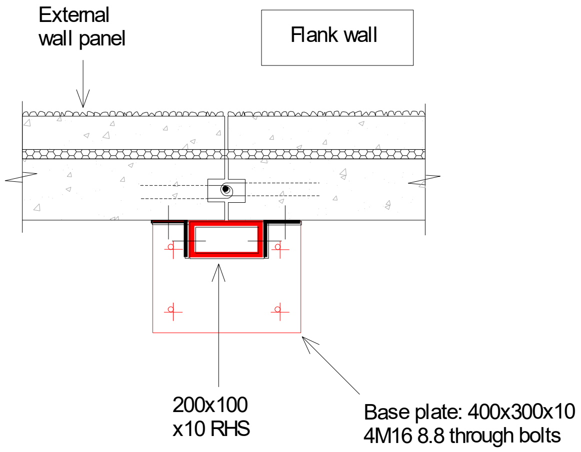
[Figure A4]: Residential Block - Long section view on the side wall panels



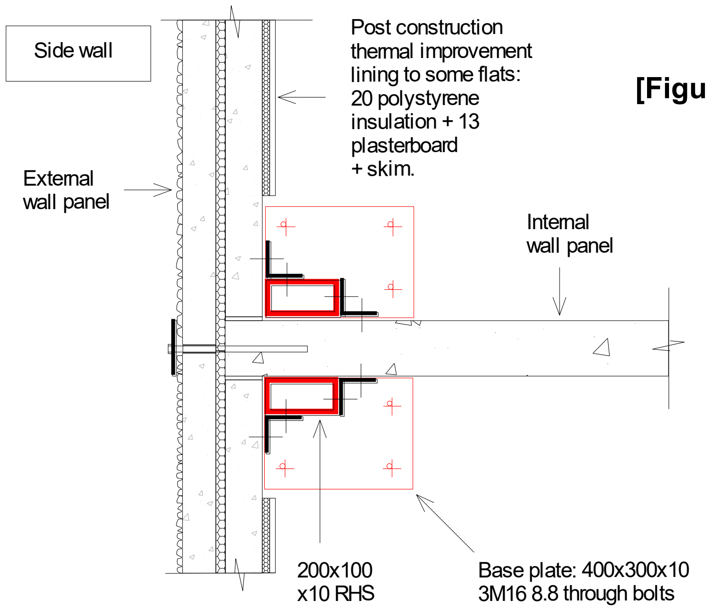
[Figure A5]: Residential Block - Short section view on the flank wall panels



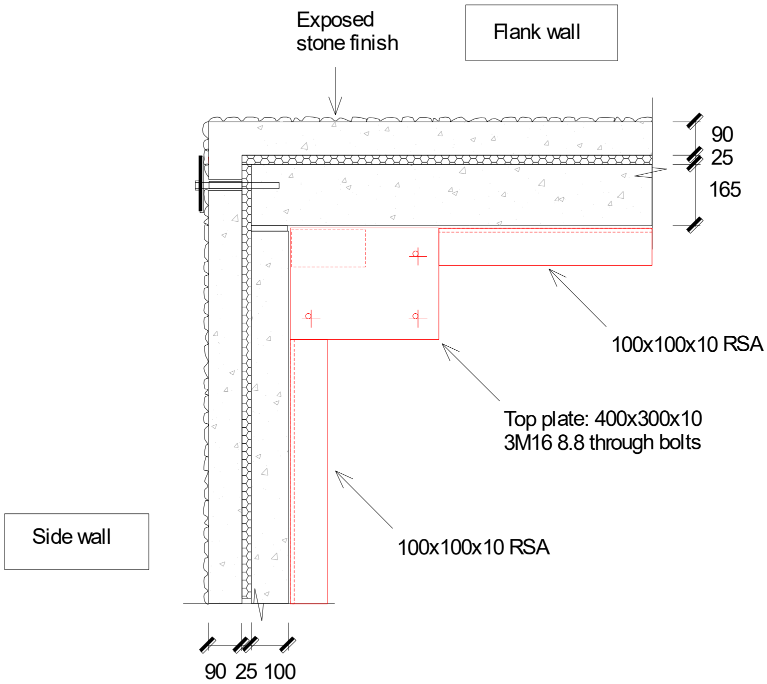
[Figure A6]: Flank to side wall connection Floor view



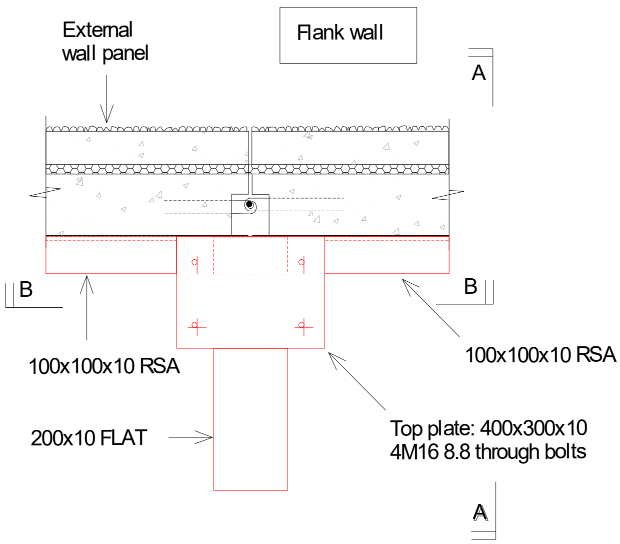
[Figure A7]: Flank wall to flank wall connection – Floor view



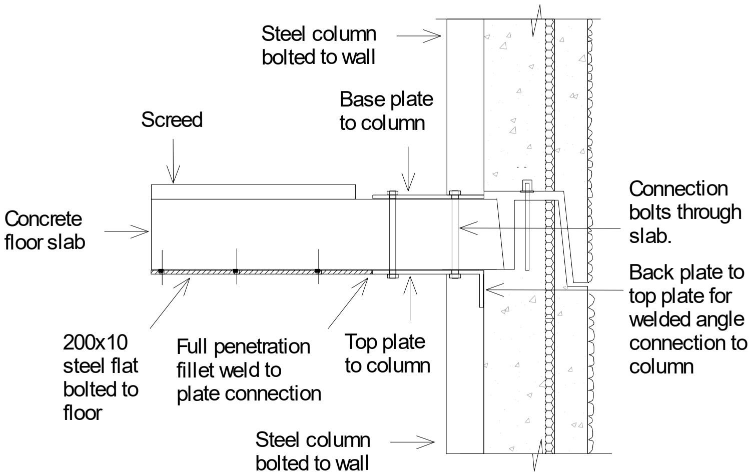
[Figure A8]: Side wall to cross wall connection – Floor view



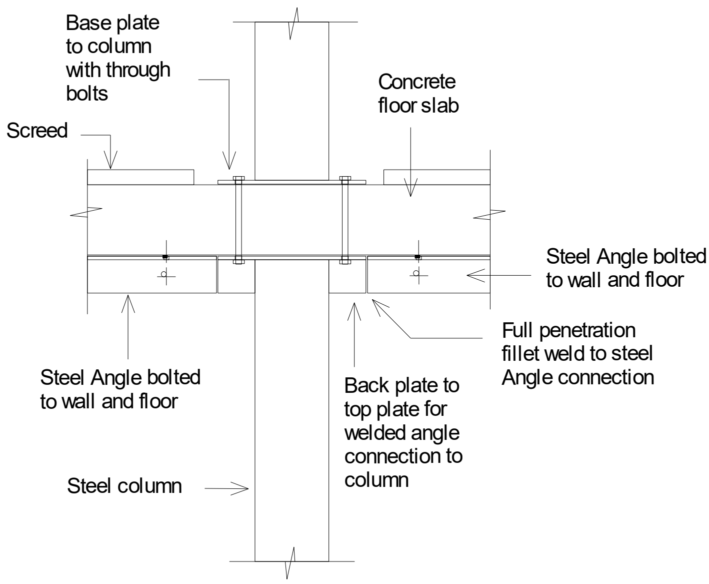
[Figure A9]: Flank to side wall connection D2 – Ceiling view



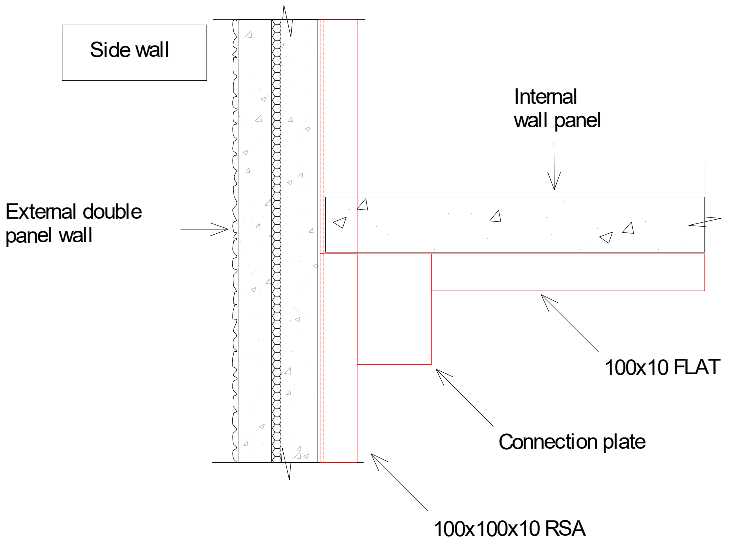
[Figure A10]: Flank wall to flank wall connection D1 – Ceiling view



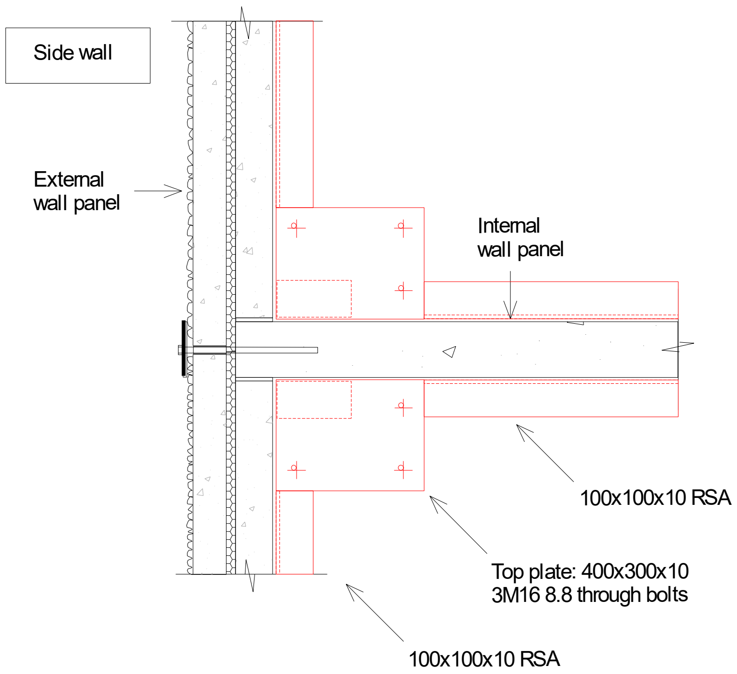
Section A – A to Figure A10
Scale 1:20



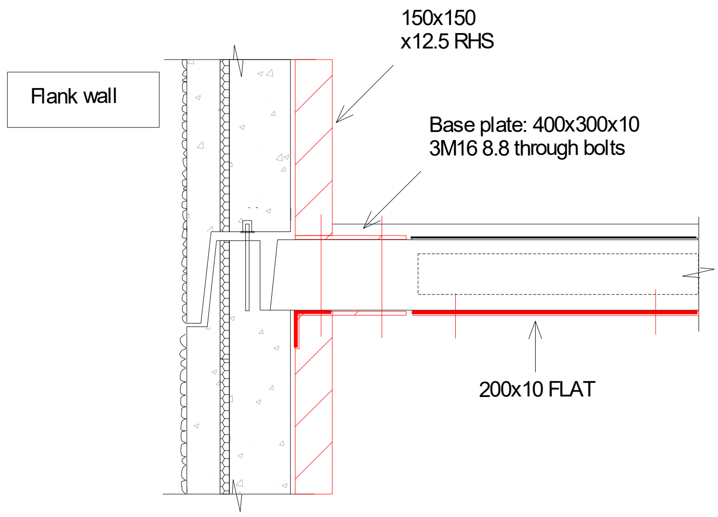
Section B – B to Figure A10
Scale 1:20



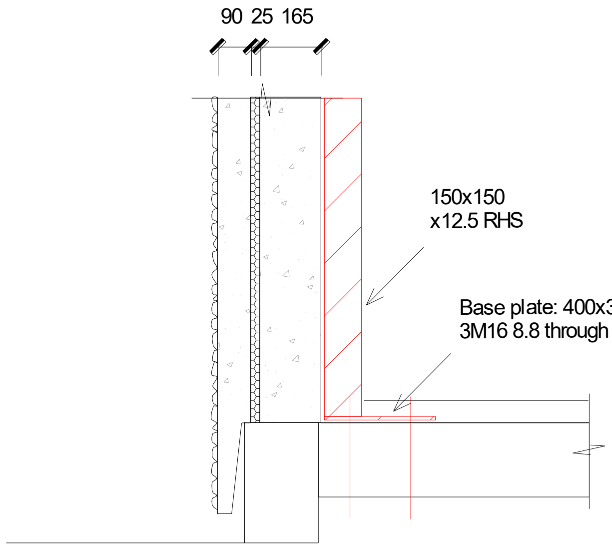
[Figure A11]: Side wall to partition wall connection D4 – Ceiling view



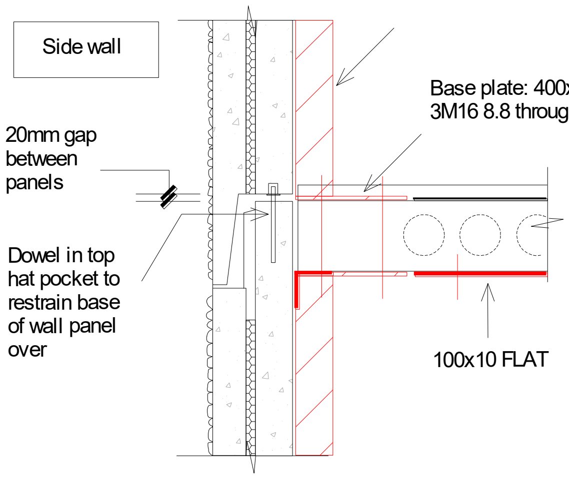
[Figure A12]: Side wall to cross wall connection D3 – Ceiling view



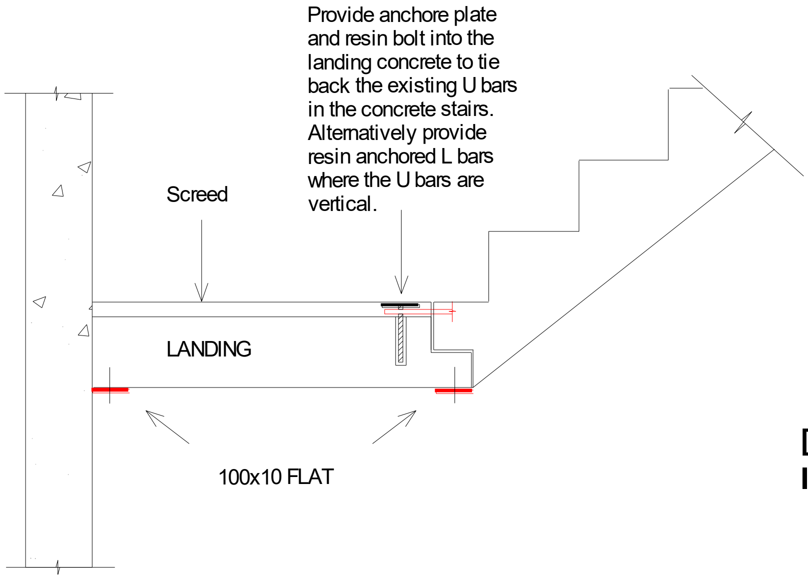
[Figure A13]: Flank wall to slab connection – Upper floors



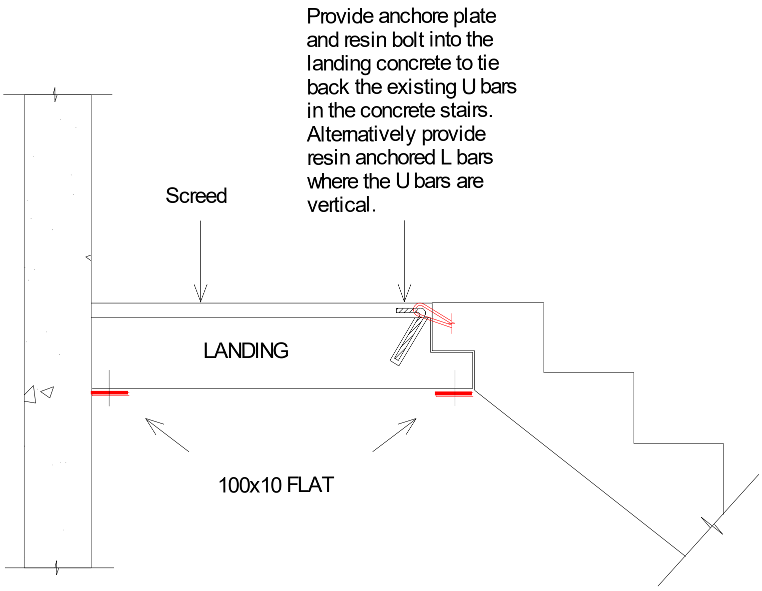
[Figure A14]: Flank wall to slab connection – Ground floor



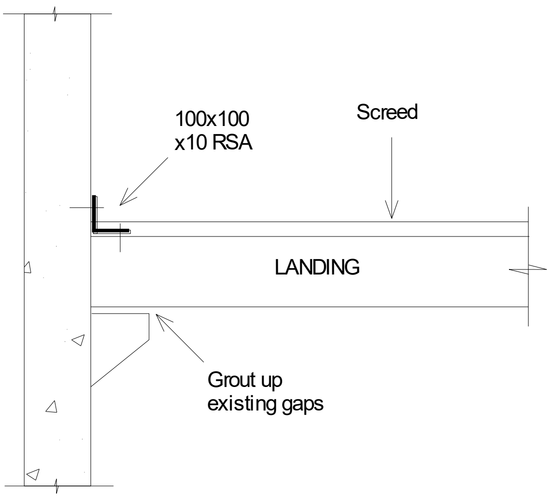
[Figure A15]: Side wall to slab connection



[Figure A16]: Stairs-to-landing connection



[Figure A17]: Stairs-to-landing connection



[Figure A18]: Landing connection

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