6 Structural Intrusive Condition Survey

6.1 Synopsis

6.1.1 Leicester City Council (LCC) commissioned WYG to undertake a Condition Survey and Options Appraisal for Goscote House, Leicester in May 2016 (Ref A). The survey included a visual non-intrusive structural inspection of 28No flats and recommended further intrusive works in order to gain an appreciation of the condition of the LPS (Large Panel System) concrete structure and the wall and floor tying provision/details adopted. This report outlines the findings of this intrusive inspection and includes a commentary on the structural condition.

6.1.2 Opening up works were undertaken on 14th to 21st Nov 2016 on various floors internally, both at high and low level. All internal opening up works were undertaken in vacated flats, office areas or plant rooms. The internal breaking out, sampling and testing work was undertaken by Kiwa CMT Ltd. External rope access work was carried out by HRS Services Ltd, all under the supervision of WYG Engineering.

6.1.3 The intrusive work focused upon the assessment of the precast wall panel/floor panel joints and in particular the presence of 'dry pack' mortar at the base of the walls, and reinforcement tying between the panels to provide stability and robustness measures. The dry pack mortar appeared to be present and well compacted, although not provided for the full width of the panel in every location. From the load assessment it is concluded that the dry-pack mortar is not overstressed.

6.1.4 The exposures reveal tying reinforcement between the precast wall and slab panels, and at the podium/precast interface i.e. the first floor junction. The conclusion is that structural stability is maintained and joints are effectively tied in respect of robustness.

6.1.5 Ref A identified localised areas of reinforcement corrosion and spalled concrete, where it is evident certain areas have already been subject to patch repair. In reinforced concrete the rusting of steel and the resulting spalling of surface concrete is principally caused by chloride contained in the concrete or carbonation of the concrete, and sometimes both. In Goscote House the test results indicate a limited MODERATE risk of corrosion in the external wall panels, due to chloride in the original mix, rather than carbonation. This is borne out by the typically localised spalling which is limited to the external elevations (Ref A). Additionally, there are localised areas of exposed corroded reinforcement where no remedial works have been undertaken to date. Moving forward consideration should be given to the repair of these local areas of spalled concrete and exposed reinforcement.

6.1.6 High Alumina cement does not appear to be present in the concrete of Goscote House.

6.1.7 A petrographic investigation to determine the presence of Alkali-Silica Reaction (ASR) indicated no evidence of a reaction between the cement paste and aggregate consistent with ASR.

6.1.8 Results indicate reasonably high compressive strengths of concrete ranging from 63.7 N/mm2 in insitu concrete podium, to 47.6 N/mm2 in an internal precast panel.

6.1.9 Moving forward it is understood a reconfiguration of the internal layout is being considered to create larger 2/3/4 bed flats. The previous recommendations (Ref A Page 22/23) should be followed in this respect, by limiting structural works in a similar manner to the reconfiguration already carried out on the 15th, 17th 19th floors, where five flat units have been created from the original seven per floor.

6.1.10 It must also be highlighted that any proposals for an over-cladding of the structure should be subject to structural review in respect of loading and fixings etc.

6.1.11 Consideration should be given to the repair of the local areas of spalled concrete as identified in Ref A together with local areas of corroded reinforcement identified during this inspection. Additionally, and outside the remit of concrete repairs, works also highlighted in Ref A included attention to areas of loose render, fixings of roof balustrades and handrail provision at roof level together with the strengthening/replacement of a steel beam supporting the water tank at high level plant room.

6.2 Introduction

6.2.1 Within a wider project remit, Leicester City Council (LCC) commissioned WYG to undertake a Condition Survey and Options Appraisal for Goscote House, Leicester in May 2016 (Ref A). The survey included a visual non-intrusive structural inspection of 28No flats and recommended further intrusive works in order to gain an appreciation of the condition of the LPS (Large Panel System) concrete structure and the wall and floor tying provision/details adopted. This report outlines the findings of this intrusive inspection and includes a commentary on the structural condition.

6.2.2 Opening up works were undertaken on 14th to 21st Nov 2016 on various floors internally, both at high and low level. All internal opening up works were undertaken in vacated flats, office areas or plant rooms. Concrete samples for laboratory testing were taken externally and internally in various locations to give an appreciation of the general condition of the concrete and reinforcement within the structure. Access was readily achieved for the internal works via existing walkways/floors. Abseil techniques were utilised for the testing and sampling works carried out externally.

6.2.3 The internal breaking out, sampling and testing work was undertaken by Kiwa CMT Ltd. External rope access work was carried out by HRS Services Ltd. Both of these activities were carried out under the supervision of WYG. Prior to the site works, site visits were made by LCC, WYG, Kiwa and HRS to appraise access restrictions and the like. Additionally, there was recognition that asbestos was present in some ceiling finishes, and this was removed locally in specific areas as identified by WYG, by the LCC asbestos team, in readiness for the commencement of site works.

6.2.4 The suggested internal testing and breakout locations as outlined in Ref A were revised due to restrictions placed upon the works within the occupied building, and work areas were subject to availability of vacated tenanted flats/offices. At the time of testing 3No flats within Goscote House were vacated and utilised for the internal investigation works. Other areas included disused offices and plant rooms on first and ground floors.

6.2.5 Any previous reports into the structural condition of Goscote House, such as inspections recommended by the Ministry of Housing and Local Government for LPS residential tower blocks (outlined in Ref C), and following the Ronan Point disaster in 1968, are not available. The lack of any structural information has been a driver for the concrete testing and opening up works commissioned by LCC. It should be recognised that the inspection has been undertaken in support of the previous visual inspection (Ref A) in order to assess the condition of the concrete structure and presence of robustness tying in respect of overall stability. A detailed assessment of robustness tying in respect of accidental loading (Ref C) has not been made and is outside the brief.

6.2.6 Weather conditions varied throughout the site works but generally cold temperatures were experienced with high winds and overcast cloud. On 21st November torrential rain with cold temperatures and high winds affected abseiling operations for a period.

6.3 Background / Description

6.3.1 The description of the structure together with a resume of opening works required is included in Ref A and Section 5 of this report and is not repeated in full here, but in brief Goscote House is a 24 storey residential tower block, 66m high, constructed 1972-73. The structure comprises an insitu reinforced concrete ground floor podium structure and precast concrete construction above. The structure is a Large Panel System (LPS) structure constructed by Taylor Woodrow Anglian (TWA) (confirmed within Ref B).

6.3.2 The structural plan layout repeats for the full height of the building and comprises load bearing precast concrete walls above the 1st floor podium level. The structural layout on ground floor differs from those above due to the inclusion of garage and plant rooms and an access route into the building from the south-west.

6.3.3 The upper floors comprise precast floor slab panels supported by the external and internal wall panels. The joints between the floor slabs are visible in the soffit of all flats inspected, with a typical slab width of 2700mm.

6.3.4 Foundation and ground floor slab details of the building are unknown.

6.3.5 Externally, the load bearing precast wall panels are fully overclad with ribbed cladding panels.

6.3.6 The previous inspection (Ref A) identified localised areas of exposed and corroded reinforcement from ground to 16th floor along with areas of previous patch repairs. Some cracking of plaster finish was noted in two flats internally. Other items included missing fixings on perimeter balustrades at rooftop level, missing edge protection adjacent to a CAT ladder and cracking and delamination of render to external walls.

6.4 Observation and Testing

6.4.1 Opening Up Works

6.4.1 The opening up works focused on two aspects considered to be critical in an LPS structure (as identified in Ref A); the presence and condition of the dry-pack mortar at the base of load bearing panels and the robustness tying reinforcement between precast wall and slab joints.

6.4.2 It is understood the quality of the dry-pack mortar joints was highly variable in construction of this type and breaking out and inspection of the mortar joints was undertaken to assess its presence and quality. The process involved breaking out the dry-pack mortar across the base of the precast wall panel. Exposures were made in 9No. locations across 3 levels and the dry-pack visually assessed and width and depth measured locally. Samples were taken for laboratory testing to assess the constituents of the mortar itself; results are included in section 6.4.2 of this report.

6.4.3 Ref B highlights the specific requirement for horizontal and vertical robustness tying reinforcement and identifies Goscote House as a Type B form of TWA structure where tying reinforcement between the precast wall and slab panels with an infilled insitu concrete joint would be anticipated. The process for checking the tying reinforcement comprised opening up at the wall – soffit joint to expose the reinforcement within the insitu joint. This visual indication together with radar scanning of the local area (Hilti radar scanning) gives an indication of the presence and arrangement of the tying reinforcement.

6.4.1.1 Panel Joints - Dry-Pack Provision

6.4.1.1.1 All breakout locations at low and high level are indicated on WYG drawing A095161-51-S-100B (Appendices B)

6.4.1.1.2 Particular attention was focused upon the dry packing at the base of the first floor panels which bear onto the ground floor podium, which by definition support the maximum loading from the 20No floors above, estimated at approximately 125 tonnes/m (factored).

6.4.1.1.3 Detailed observations by Kiwa CMT are available within their report. The exposures are summarised as follows;

- **Location L1** located at first floor along an external wall. 3No dowel bars at 600mm c/c within the wall panel and 2No service voids (see photo 1). Dowels 26mm dia. plain round bars located at the centre of the precast panel width (see photo 2 typical). Dry pack at base of panel 30mm deep and 80mm wide across the 150mm wide panel. Service voids are rebates cast into the precast wall panel 50mm deep by 100mm in length. The voids are for electrical services embedded within the wall panel and filled with plaster material. Breaking out the dry-pack broke through to the external overclad panel, which is separated from the inner panel by compressible fill material.
- **Location L2** located at first floor along an internal wall panel exposed a steel box section within the precast panel and levelling dowel bolted down with a 40mm dia. Nut and 10No pack washers (see photo 3). The box section contained a slotted hole on the bottom face to assist locating the levelling dowel. A secondary threaded bar passes through the top flange of the box section and into the remainder of the precast wall panel (see photo 4). A service void rebate was exposed and the dry-pack fill broken out to reveal the panel width as approx 240mm. Dry-pack measured 30mm deep for the full width beneath the precast wall panel.

- **Location L3** located at first floor along an internal flank wall revealed 180x130mm rebate filled with dry-pack material. The precast panel contained a 65mm dia. coupler protruding 100mm from the podium level slab and a 40mm dia. threaded dowel bar into a formed void within the precast panel (see photo 5). The coupler and dowel are located at the centre of the width of the wall panel. The dry-pack measured 25mm in depth and is full width across the width of the panel (250mm wide).

- **Location L4** located at first floor along an external wall panel. The breakout exposed 2No dowels bars at 600mm c/c within the external wall panel and 1No service void (see photo 6). The dowel bars were 26mm dia. plain round bars and set back from the inside face by 57mm. The dry-pack appeared in good condition and measured 20mm deep. The width of the dry-pack is approximately 50mm from each face, giving 100mm overall bearing width. Next to the service void the width measured approx 120mm in total.
- **Location L5** located on the 6th floor along an external wall panel. The breakout exposed 2No 26mm dia. plain round dowel bars at 600mm c/c within the external wall panel (see photo 7) set back from the inside face by 65mm. The precast panel measured 150mm wide. Dry pack in good condition measuring 25mm deep along the length of the panel and full width across the 150thk panel. A single service void was exposed adjacent to the levelling dowels and appeared to be partly filled with plastic packaging. The overclad panel was revealed and exposed a plastic fill type fill material (see photo 8).
- **Location L6** located on the 6th floor along an internal wall panel revealed 2No 26mm dia. plain round dowel bars at 1200mm c/c. with evidence of a third bar midway between the two i.e. giving tying at 600mm c/c. The dowels were set back ~105mm from the panel face. Presence of plastic packing material meant assessing the width and condition of dry-pack was difficult. Width of wall panel approximately 225mm. Dry pack appeared to be present full width across the panel.
- **Location L7** located on the 16th floor along an external wall panel. Breakout exposed a service void 50mm deep by 160mm long, along with 2No plain round dowel bars, 26mm dia. at 600mm c/c. The dowel bar runs in a void formed by a 60mm dia. ribbed, metal pipe embedded within the panel (see photo 9). Possibly a typical detail throughout the building. The LHS dowel bar centre was measured to be 2020mm from the external end panel inside face. Dry-pack in good condition and 25mm in depth and installed across the full width of the 150mm wide panel including the infill to the service void (see photo 10).
- **Location L8** located on the 16th floor along an external wall panel. 2No plain round dowel bars, 26mm dia. at 600mm c/c (see photo 11), with a service void, infilled dry-pack mortar. The service void measured 100mm deep into the panel from the inner face. The dry-pack

was 35mm in height and was consistent across the full width of the panel (see photo 12). The void between the overclad panel and the load-bearing panel was filled with compressible air-filled packing material (see photos 12 & 13).

- **Location L9** located on the 16th floor along an internal wall panel. The breakout exposed a large 230x200mm rebate within the wall filled with a combination of dry-pack and plaster (see photo 14). Within the rebate a threaded 26mm dia. dowel bar protruded out of the floor slab and coupled into a secondary 42mm dia. bar running vertically into the panel within a cast-in void. A second threaded dowel bar was exposed at 600mm c/c further along the wall (see photo 15). This is the only breakout where threaded dowel bars were observed. Three service voids were exposed within the breakout, all 190mm apart. The dry-pack appeared full width across the 240mm wide panel and measured 35mm in depth. Areas of reduced dry-pack width at the service voids 50-60mm from each side. Shims left within the void between slab and underside of precast panel (see photo 16) were not loaded.

6.4.1.2 Wall/Slab Junctions - Tying Reinforcement

6.4.1.2.1 All breakout locations at low and high level are indicated on WYG drawing A095161-51-S-100B (Appendices B).

- **Location U1** located on the first floor along an internal wall panel exposing the bearing and insitu infill between the floor slab and the top of the wall panels. 15mm slab bearing with no bedding material present with the slab bearing directly onto the wall panel (see photo 17). The breakout was not large enough to expose any tying rebar.
- **Location U2** located on the first floor along an external wall panel. The breakout exposed several reinforcing bars within the slab, wall panel and insitu infill concrete (see photos 18 & 19). Tying reinforcement both horizontally and vertically present within the concrete infill. The horizontal tie appears to be 10mm dia. plain reinforcing bar, hooking around the main reinforcement (possibly 16mm dia.) within the infill joint; the anchorage length into the slab is unknown. The vertical tie (10mm dia. plain bar) protrudes from the wall panel into the infill; it is unclear whether there is a hook on the end of the bar within the infill; anchorage length into the wall panel is unknown. The slab main reinforcement was 16mm dia. in the span direction with 10mm dia. distribution bars with 15mm cover as the outer layer. 16mm dia. reinforcement bar with 30mm cover in the wall panel. No vertical reinforcement was

uncovered in the wall panel. The bearing of the slab panel onto the wall looks to be 15mm. The distance to centre of horizontal tying rebar was 460mm from the edge of the slab panel.

- **Location U3** located on the 16th floor along an internal wall panel. The breakout exposed horizontal tying reinforcement (10mm dia. plain bar) which appeared to continue through the concrete infill and into the adjacent precast slab panel (see photo 20 & 21). This detail is unconfirmed but assumed due to the extent of breakout with no evidence of the rebar hooking around secondary bars as was demonstrated in location U2. No vertical tying rebar was revealed; however, it is assumed the breakout avoided further rebar given its spacing. Main reinforcing bars exposed within the floor slab panel with 16mm dia. main steel and 8mm dia. plain round distribution bars with 20mm of cover. Reinforcement in the precast wall panel was not exposed. The bearing of the slab onto the wall panel measured as 45mm. The distance to centre of horizontal tying rebar was 460mm from the edge of the slab panel.
- **Location U4** located on the 16th floor along an external wall panel. The breakout exposed both vertical and horizontal tying reinforcement within the joint, together with the main reinforcement present within the slab panel (see photo 22 & 23). Tying reinforcement measured 10mm dia. plain bars tied into the insitu concrete joint. The slab reinforcement appears to be 16mm main bars and 8mm distribution bars as elsewhere, with 25mm cover in the minor direction. The horizontal and vertical ties are located 255mm c/c apart within the breakout, with the horizontal tie 1360mm from the slab panel edge. The bearing of the slab onto the wall panel was measured as 20mm. The breakout did not expose any main reinforcement within the wall panel.

6.4.2 Concrete Testing

The concrete testing works comprised;

(i) Half-cell potential

(ii) Concrete resistivity survey

(iii) Chloride content analysis; via concrete dust sampling

(iv) High alumina cement (HAC) analysis

(v) Core samples for density & compressive strength

(vi) Core samples for Alkali-Silica Reaction (ASR)

(vii) Cement content analysis; via concrete dust sampling

(viii) Dry-pack mortar analysis; via dust sampling

(ix) Cover to reinforcement survey

(x) Carbonation depth testing

(xi) Hilti radar scanning

Samples were taken internally and externally from various floors and elevations to attain a representative sample without mass disruption or a prolonged period on site.

The extent of the sampling and testing locations are identified on WYG drawings A095161-51-S-100B/105B (Appendices B)

6.4.2.1 Interpretation of Concrete Testing Results - Half –Cell Potential

6.4.2.1.1 Half-cell potential is used to give an indication of the condition of the steel reinforcement below the concrete surface. However, the nature of this method means that it does not provide any information on the rate of deterioration. Half-cell potentials were carried out on 4 external panel surfaces on different elevations of the building using 100mm spacing grid of 10 x 10 where possible. Half-cell potential measurements can give an indication of the probability of corrosion to the steel reinforcement. Table 1 identifies the likelihood/probability of corrosion for measured value of half-cell potential.

Half-cell Potential Measurement	Probability of Corrosion
Less negative than -200mV	5% (Low risk)
Between -200mV and -350mV	50% (Moderate risk)
More negative than -350mV	95% (High risk)

Table 1; criteria for corrosion risk in reinforcement

(Taken from table 26, Concrete Society Technical Report 54; 'Diagnosis of deterioration in concrete structures' Ref D)

Testing Results

6.4.2.1.2 Half-cell potential measurements, for each of the tested areas are presented in full within HRS's report (Appendix B). Table 2 (below and reproduced from HRS's report) identifies the percentage of half-cell measurements falling within a specific range of corrosion risk. The tested panel locations are shown on WYG drawing A095161-51-S-105B.

Test Area No	Resistivity (kΩ cm)	Half-cell Potential Measurements (% falling in the range of)		
		Less negative than -200mV	-200mV to -350mV	More negative than -350mV
HCT 01 (NE face, level 16)	42.0	100	0	0
HCT 02 (SE face, level 4)	101.7	100	0	0
HCT 03 (SW face, level 16)	69.3	100	0	0
HCT 04 (SE face, level 1)	82.9	100	0	0

Table 2; Half-cell potential measurements indicating the likelihood of corrosion.

6.4.2.1.3 It can be seen that all locations tested indicated a low risk of corrosion to the reinforcement with no readings falling below the less negative than -200mV threshold.

6.4.2.2 Interpretation of Concrete Testing Results - Concrete Resistivity Survey

6.4.2.2.1 Concrete resistivity provides an indication of the likely corrosion rate once corrosion has been initiated, based upon the electric current generated by the corrosion. Table 3 identifies the relationship between the corrosion probability and measured resistivity.

Resistivity	Likely Corrosion Rate
> 20 kΩ cm	Negligible Probability
20 – 10 kΩ cm	Low Probability
10 – 5 kΩ cm	High Probability
< 3 kΩ cm	Very High Probability

Table 3; criteria for corrosion probability in reinforcement

(Taken from table 27, Concrete Society Technical Report 54; 'Diagnosis of deterioration in concrete structures' Ref D)

Testing Results

6.4.2.2.2 Concrete resistivity results for each area tested are presented within HRS's report (Appendix B) but are reproduced within table 2 (above).

6.4.2.2.3 All tested areas demonstrated a high resistivity indicating there is negligible probability of corrosion occurring within the tested area.

6.4.2.3 Interpretation of Concrete Testing Results - Chloride Content analysis

6.4.2.3.1 Chlorides can contribute to corrosion and calcium chloride may have been added to or in the original concrete mix. Its effects are not confined to the surface.

6.4.2.3.2 The test quantifies the amount of chlorides within the concrete. Dust samples for chloride-ion content were taken at sample locations using a rotary percussion drill fitted with a 25mm dia. drill bit. Samples were collected at 25mm increments up to a maximum depth of 75mm, the first 5mm of the outer sample being disregarded as non-representative.

Chloride-ion Content (highest percentage across depth range by mass of cement)	Risk of corrosion
< 0.3%	Negligible (Low)
0.3% – 1.0%	Possible (Moderate)
> 1.0%	Probable (High)

Table 4; criteria for chloride-ion content and corrosion risk in concrete

Testing Results

6.4.2.3.3 Full results of chloride content analysis are available within Kiwa CMT and HRS's respective reports (Appendix A and B). However, a summary of the results is reproduced in table 5 below. The table identified the highest percentage chloride-ion content (by mass of cement*) at the location of each sample point, across the depth range.

* - assumes 14% cement content (as stated in the respective reports)

6.4.2.3.4 Although the chloride-ion content is consistently below the high corrosion risk of 1.00%, 15 out of the 23 samples from the external elevations suggest a moderate risk of corrosion due to chlorides. In one isolated external test area AB20 with a maximum content of 1.25% was recorded. This test was carried out on the south eastern elevation at level 8, within the dust sample range obtained between 30 to 50mm. Section 5 includes a discussion on reinforcement corrosion.

Test Area	Highest % Chloride-ion content (by mass of cement*)
External Samples	
AB1 – NE, level 20	0.37
AB2 – NE, level 12	0.86
AB3 – NE, level 6	0.49
AB4 – NE, level 1	0.73
AB5 – NW, level 20	0.50
AB6 – NW, level 15	0.13
AB7 – NW, level 10	0.24
AB8 – NW, level 4	0.92
AB9 – NE, level 16	0.61
AB10 – NE, level 8	0.16
AB11 – NW, level 17	0.97
AB12 – NW, level 8	0.30
AB13 – SE, level 16	0.97
AB14 – SE, level 4	0.11
AB15 – SE, level 20	0.72
AB16 – SW, level 18	0.41
AB17 – SW, level 16	0.26
AB18 – SE, level 13	0.12
AB19 – SW, level 11	0.53
AB20 – SE, level 8	1.25
AB21 – SW, level 4	0.40
AB22 – SW, level 2	0.15
AB23 – SE, level 1	0.51
Internal Samples	
TA1 – level 1, NW office, ext wall	<0.02
TA2 – level 1, NW office, int wall	<0.02
TA3 – level 1, NW office, ext wall	<0.02
TA4 – level 1, NW office, ext wall	0.10
TA5 – level 1, S office, int wall	<0.02
TA6 – level 1, S office, ext wall	<0.02

Test Area	Highest % Chloride-ion content (by mass of cement*)
TA8 – level 1, S kitchen, int wall	<0.02
TA9 – level 1, adj lift lobby, int wall	0.25
6-TA-A – level 6, flat 67, ext wall	<0.02
16-TA-A – level 16, flat 164, int wall	<0.02
16-TA-B – level 16, flat 164, ext wall	0.07
16-TA-C1 – level 16, flat 164, ext wall	0.10
16-TA-D – level 16, flat 164, int wall	0.05
21-TA1 – level 21, flat 217, ext wall	<0.02
21-TA2 – level 21, flat 217, ext wall	0.05
21-TA3 – level 21, flat 217, int wall	<0.02
21-TA4 – level 21, flat 217, ext wall	0.05
21-TA5 – level 21, flat 217, ext wall	<0.02

* assumes 14% cement content.

Table 5; Chloride-ion content (highest percentage) (table reproduced from Kiwa CMT and HRS testing reports)

6.4.2.4 Interpretation of Concrete Testing Results – High Alumina Cement (HAC) analysis

6.4.2.4.1 Based on the date of construction of the original building, there is a possibility of HAC being used in the construction.

Testing Results

6.4.2.4.2 A total of 10 samples were taken to identify the presence of HAC within the concrete. All samples tested returned a negative result for the presence of HAC.

6.4.2.5 Interpretation of Concrete Testing Results – Core Samples for density and compressive strength

6.4.2.5.1 6No 75/100 mm dia. core samples were obtained from the structure and tested to determine the density, compressive strength and presence of any embedded reinforcement. 4No core samples were taken internally; 2No from the insitu reinforced concrete podium and 2No from internal precast wall panels. 2No core samples were taken from external (outer) wall panels.

Testing Results

6.4.2.5.2 Full results of core samples are available within Kiwa CMT and HRS's respective reports (Appendix A and B). A summary of the results is shown in table 6.

Core Samples Location	Core Diameter (mm)	Density of Core (kg/m^3)	Compressive Strength (N/mm^2)	Steel Reinforcement
Internal ground floor tenants store (Insitu concrete)	100	2397	63.7	None identified
Internal ground floor electrical room (Insitu concrete)	100	2398	60.6	12.5mm (22mm cover) 12.5mm (35mm cover)
Internal level 1 – NW office, ext wall (Precast concrete)	75	2307	47.6	12mm (17mm cover)
Internal level 16 – Flat 164, int wall (Precast concrete)	75	2342	61.5	None identified
External level 1 – SE elevation (Precast concrete)	100	2440	49.5	None identified
External level 16 – NE elevation (Precast concrete)	100	2430	49.8	6mm (60mm cover) 6mm (64mm cover)

Table 6; tested core compressive strength and reinforcement present

6.4.2.6 Interpretation of Concrete Testing Results – Alkali –Silica-Reactions (ASR)

6.4.2.6.1 2No 75/100 mm dia. core samples were obtained from the insitu reinforced concrete podium and precast wall panels.

Testing Results

6.4.2.6.2 The full test report for ASR is included within Kiwa CMT's report (Appendix A). In summary it was found there is no evidence of a reaction between the cement paste and aggregate consistent with ASR and no secondary sulphate minerals were identified.

6.4.2.7 Interpretation of Concrete Testing Results – Cement Content analysis

6.4.2.7.1 This testing assesses the quantity of cement used both in the insitu podium concrete and various precast panels and gives an appreciation of consistency of concrete generally. 10No dust samples were obtained from external and internal facades at different levels of the building.

Testing Results

6.4.2.7.2 The full results from the cement content analysis are available within Kiwa CMT's report (Appendix A). A summary of cement content samples is shown in table 7 below.

Test Area	Cement Content Percentage based on Sample
AB2 – NE, Level 12 External Sample	16.04
AB9 – NE, Level 16 External Sample	15.35
AB14 – SE, Level 4 External Sample	15.69
TA1 – level 1, NW office, ext wall Internal Sample	12.77
TA5 – level 1, S office, int wall Internal Sample	21.39
TA6 – level 1, S office, ext wall Internal Sample	17.72
6-TA-A – level 6, flat 67, ext wall Internal Sample	18.42
16-TA-B – level 16, flat 164, ext wall Internal Sample	18.91
Electrical Room G/F, int wall	14.93
Tenants Store G/F, ext wall	19.67

Table 7; sample locations and associated cement content percentage

6.4.2.8 Interpretation of Concrete Testing Results – Dry-Pack Mortar analysis

6.4.2.8.1 6No dust samples were obtained from the dry-pack mortar following the localised breakout internally. The dust samples are used to determine the constituents of the mortar and thus give an appreciation of the compressive strength.

Testing Results

6.4.2.8.2 The full results from the dry-pack mortar analysis are available within Kiwa CMT's report (Appendix A). A summary of the mortar samples is shown in table 8 below.

6.4.2.8.3 The generally consistent mix proportions of 1:0:2 of cement: lime: sand suggests a relatively high compressive strength.

Test Area	Cement Content Percentage based on Sample	Composition by Volume Cement : Lime : Sand
TA1 – level 1, NW office, ext wall	24.4	1:1:2
TA2 – level 1, NW office, int wall	36.0	-
TA6 – level 1, S office, ext wall	37.0	-
TA9 – level 1, adj lift lobby, int wall	30.4	1:0:2
6-TA-A – level 6, flat 67, ext wall	32.1	1:0:1.5
16-TA-B – level 16, flat 164, ext wall	35.5	1:0:1.5

Table 8; sample locations, cement content and mortar mix proportions of dry-pack

6.4.2.9 Interpretation of Concrete Testing Results – Cover to reinforcement survey

6.4.2.9.1 Cover and position of reinforcement was determined at locations specified using a scribe CPM pulse induction covermeter. Bars were located in the outer layer of reinforcement and the minimum depth of cover recorded.

Testing Results

6.4.2.9.2 Results of the covermeter survey are available within HRS's report (Appendix B). However, a summary of the survey results is presented within table 9 below. The covermeter survey was only undertaken externally alongside the carbonation depth testing.

Test Area	Minimum Cover Depth (mm)	Depth of Carbonation (mm)
AB1	Exposed steel	1
AB2	26	1
AB3	1	< 1
AB4	Exposed steel	1
AB5	18	2
AB6	13	1
AB7	21	1
AB8	23	< 1
AB9	25	2
AB10	45	< 1
AB11	35	1
AB12	28	< 1
AB13	20	4
AB14	11	4
AB15	21	2
AB16	13	< 1
AB17	36	1
AB18	14	< 1
AB19	45	< 1
Test Area	Minimum Cover Depth (mm)	Depth of Carbonation (mm)
AB20	14	< 1
AB21	35	3
AB22	38	< 1
AB23	40	< 1

Table 9; Minimum measure cover to reinforcement and measured carbonation depth

Test areas AB1, AB2 and AB4 have locally exposed corroded reinforcement. Photograph 24 shows the localised area of exposed reinforcement, which appears to be a horizontally running bar with exposure at the trough of the rib within the panel.

6.4.2.10 Interpretation of Concrete Testing Results – Carbonation depth testing

6.4.2.10.1 Carbonation is the process of carbon dioxide from the air infiltrating the concrete which can lead to reinforcement corrosion depending on the depth of carbonated concrete and depth of steel.

Testing Results

6.4.2.10.2 Full results are available within HRS's report (Appendix B) but they have been added to table 9 above. The table identifies the maximum depth of carbonation at each testing location across the structure.

6.4.2.11 Interpretation of Concrete Testing Results – Radar Scanning

6.4.2.11.1 Radar scanning is a non-destructive test to determine reinforcement arrangement within concrete elements. The Hilti PS1000 X-Scan scanner was used internally and externally on precast wall and slab panels to determine the reinforcement arrangement. The radar scanner can detect reinforcement arrangement, spacing, diameter and depth in a 600x600mm testing area.

Testing Results

6.4.2.11.2 Full results of the radar scanning locations are presented in the form of 2D and 3D images within HRS's report (Appendix B).

6.5 Discussion and Conclusions

The extent and locations of the testing is indicated on drawings in Appendices B. Internal locations varied from the potential locations indicated on 'pre-site works' drawings due to factors such as accessibility, timeframe and ongoing feedback from results from breakout areas. Breakout testing within tenanted flats was excluded due to time constraints. External testing locations were generally as proposed although subject to progress in to high winds.

6.5.1 Cement Content and Dry-Pack Mortar Content

6.5.1.1 The cement content analysis found the cement content varied dependent upon the concrete elements within the structure. A majority of results were above 15% cement content from the samples tested, with the highest of 21.39% tested on an internal wall on level 1. 15% appears to be average cement content for a rich cement mix, which results in a high compressive strength of concrete. Additionally, a higher cement content aids in the protection of the concrete from carbonation penetration, which is apparent since the maximum carbonation depth is 4mm, as mentioned above.

6.5.1.2 The exposure at the base of the wall panels revealed the presence of dry pack mortar which appears to be of a reasonable quality and presented no surface cracking and did not crumble easily when drilled. Although the mortar may not be provided for the full width of the panel in every location, it appeared to be well compacted and maintains a uniform contact area between the wall and slab panels, distributing the upper floor loadings to the panel below.

6.5.1.3 The mortar was analysed for constituents to determine the compressive strength of the mortar. 6No tests were undertaken and returned similar results, with a cement : lime : sand ratio of 1:0:1.5 on average. According to Kiwa CMT's results, this mixture type is stronger than the designation mortar type i, which corresponds to compressive strength class M12 (BS EN 1996-1-1). Class M12 is considered to have a compressive strength of 12 N/mm2 at 28 days.

6.5.1.4 Based on the areas where dry-pack mortar samples were taken a calculation to determine the maximum compressive stress at the sample point has been undertaken. This results in a stress based on worst case width of dry-pack in each test location. From these calculations, the highest stress encountered within the dry-pack is approximately 11.5 N/mm2.

6.5.1.5 From these results, it is concluded that the dry-pack mortar is not overstressed

6.5.2 Robustness/Wall and Slab panel tying

6.5.2.1 The exposures reveal tying reinforcement between the precast wall and slab panels, and at the podium/precast interface i.e. the first floor junction. Whilst no detailed calculations have been undertaken in report of the quantity of reinforcement and stability loading, the conclusion is that structural stability is maintained and joints are effectively tied in respect of robustness.

6.5.3 Reinforcement Corrosion

6.5.3.1 In reinforced concrete the principal cause of reinforcement corrosion is either chloride contained in the concrete or carbonation of the concrete. Both can lead to rusting of steel and the resulting spalling of surface concrete.

6.5.3.2 The test results generally point to there being a MODERATE risk of corrosion in the external wall panels, due to chloride in the original mix, rather than carbonation, since the carbonation depth is relatively low compared to the concrete cover to the steel. This is borne out by the typically localised spalling which is limited to the external elevations (Ref A). Consideration must be given to the repair of the local areas of spalled concrete as identified in Ref A.

6.5.3.3 In isolated locations in the external ribbed panels, the cover survey revealed minimal (or no) cover to horizontal reinforcement in the troughs of the ribs. These areas will also require remedial repairs moving forward.

6.5.3.4 It is evident certain areas have already been subject to patch repair. There should be recognition that workmanship used in this type of repair work is critical in respect of longevity of repair.

Areas that have been previously patch repaired are possibly prone to corrosion in the future, but the previous repairs were not inspected during the site works.

6.5.3.5 A cover meter survey was not undertaken internally, but carbonation depths were consistently measured at approximately 3mm throughout several levels of the building including the insitu podium level. A non-destructive radar scan was undertaken at 16 locations internally across 4 levels to ascertain steel arrangement, reinforcement size and cover levels. The cover level varied depending on the concrete element type being measured. Wall and floor locations were measured in both precast and insitu podium concrete.

Precast wall element cover varied from 20 – 100mm

Precast floor element cover varied from 25 – 40mm

Insitu podium wall element cover varied from 20 – 30mm

Insitu podium floor element cover varied from 20 – 110mm

6.5.4 Concrete Quality

6.5.4.1 HAC determination was undertaken on 10No samples both internally and externally and a negative result was returned for each sample.

6.5.4.2 A petrographic investigation was undertaken by Kiwa CMT to determine the presence of ASR in 2No cores and both samples returned no evidence of a reaction between the cement paste and aggregate consistent with ASR.

6.5.4.3 The compressive strength of the 2No insitu podium level concrete samples were high at 60.6 and 63.7 N/mm2. These samples were taken from an internal and external wall at ground floor level. Results from precast panels indicate compressive strengths ranging from 61.5 N/mm2, to 47.6 N/mm2 in an internal panel. 2No cores were taken via abseil on the external façade which returned similar results of 49.5 and 49.8 N/mm2.

6.5.5 Refurbishment Proposals - Structural Commentary

6.5.5.1 Moving forward it is understood a reconfiguration of the internal layout is being considered to create larger 2/3/4 bed flats. The previous recommendations (Ref A Page 22/23) should be followed in this respect, by limiting structural works in a similar manner to the reconfiguration already carried out on the 15th, 17th 19th floors, where larger 'cluster' flats have been created combining three flat units from the original seven per floor.

6.5.5.2 It must also be highlighted that any proposals for an over-cladding of the structure should be subject to structural review in respect of loading and fixings etc.

6.5.5.3 With regard to repair works, and as noted in 6.5.3 above, consideration must be given to the repair of the local areas of spalled concrete as identified in Ref A.

6.5.5.4 Additionally and outside the remit of concrete repairs, works also highlighted in Ref A (repeated within Appendix A) include attention to areas of loose render, fixings of roof balustrades and handrail provision at roof level together with the strengthening/replacement of a steel beam supporting the water tank at high level plant room.

Appendices A – Structural Photographs

Photograph 2

Breakout L1

Dowel bar and service void rebate (typical)

Photograph 2

Breakout L1

Dowel bar and service void rebate (typical)

Photograph 3

Breakout L2

Levelling dowel, washers and nut in steel box section

Photograph 4

Breakout L2

Threaded and bolted bar connecting into box section

Photograph 2

Breakout L1

Dowel bar and service void rebate (typical)

Photograph 5

Breakout L3

Threaded coupler in formed rebate in 250mm internal precast wall

Photograph 6

Breakout L4

Photograph 7

Breakout L5

Exposed dowel bar

Photograph 8

Breakout L5

Breakout exposed plastic fill material behind wall panel

Photograph 9

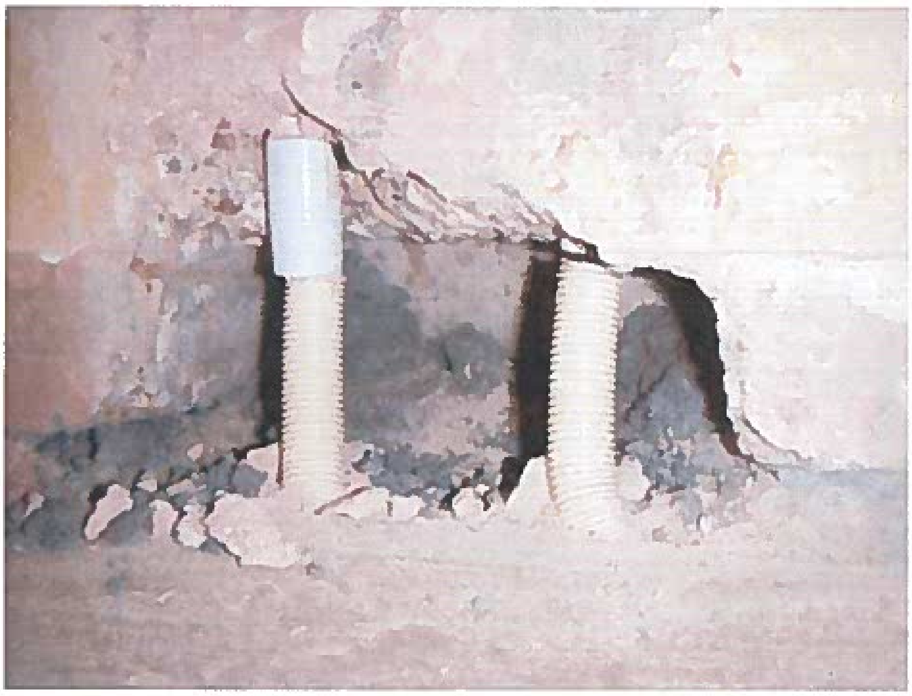
Breakout L7

Metal pipe forming void for levelling dowels

Photograph 10

Breakout L7

Service void filled with dry-pack material.
Material fills full width of panel



Photograph 11

Breakout L8

Photograph 12

Breakout L8

Dry-pack mortar in good condition and across the panel width

Photograph 13

Breakout L8

Air-filled compressible fill material at rear of panel

Photograph 14

Breakout L9

Coupler system at floor 16 within internal wall panel

Photograph 15

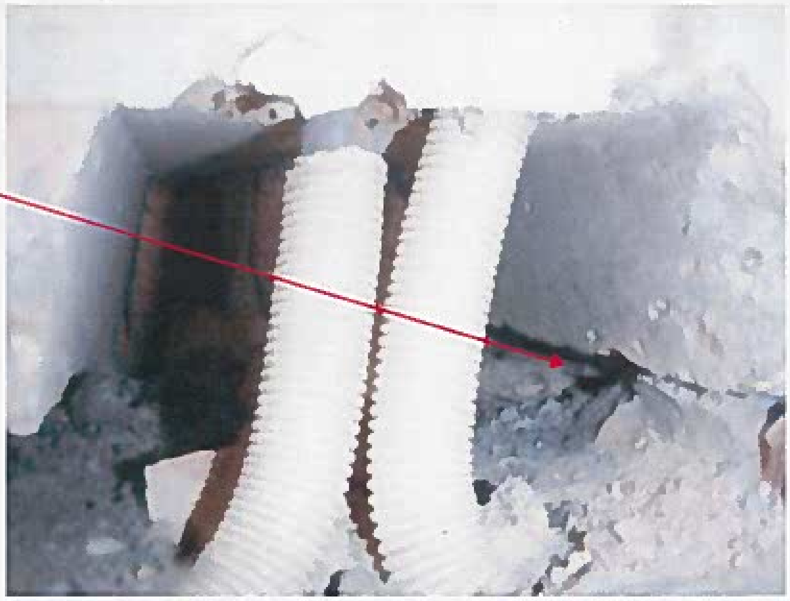
Breakout L9

Threaded dowel bar protruding into precast wall panel

Photograph 16

Breakout L9

Shim observed in void beneath panel



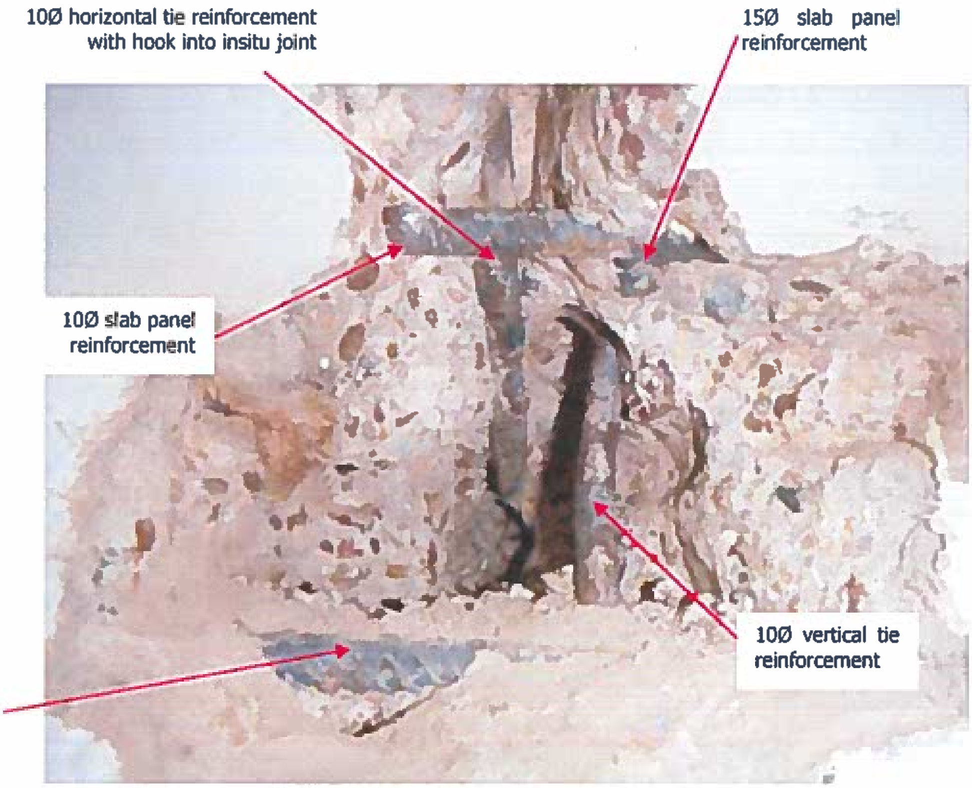
Photograph 17

Breakout U1

Ceiling level breakout showing the different concrete elements (shades) within the joint

Photograph 18

Breakout U2 tying reinforcement at wall – soffit joint at first floor

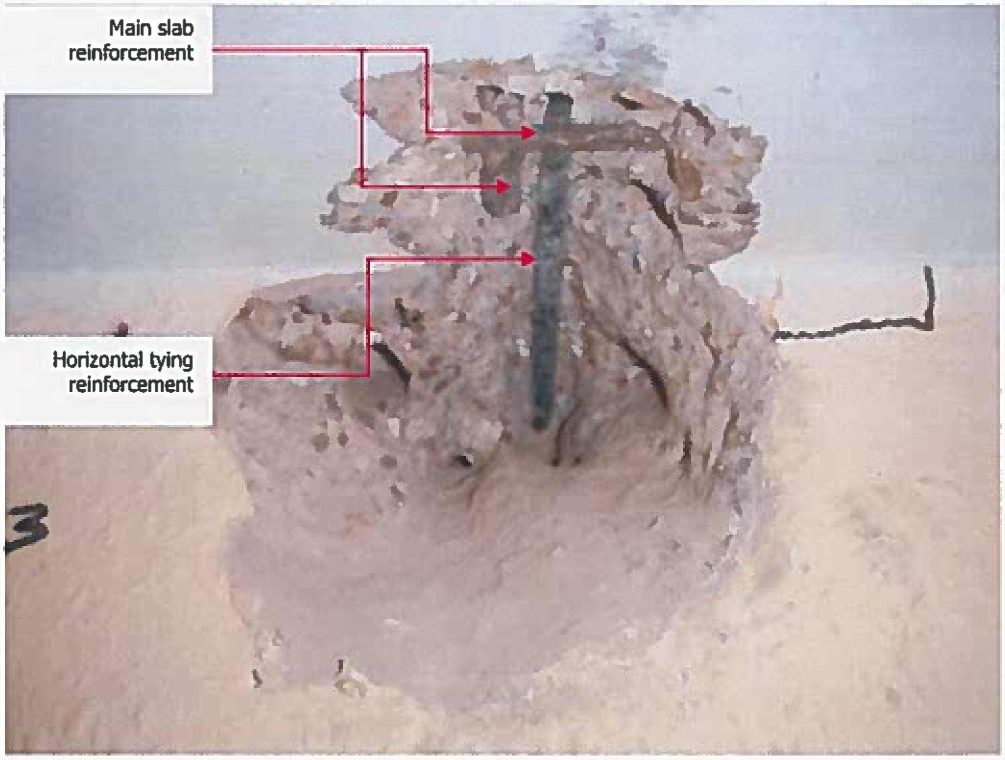


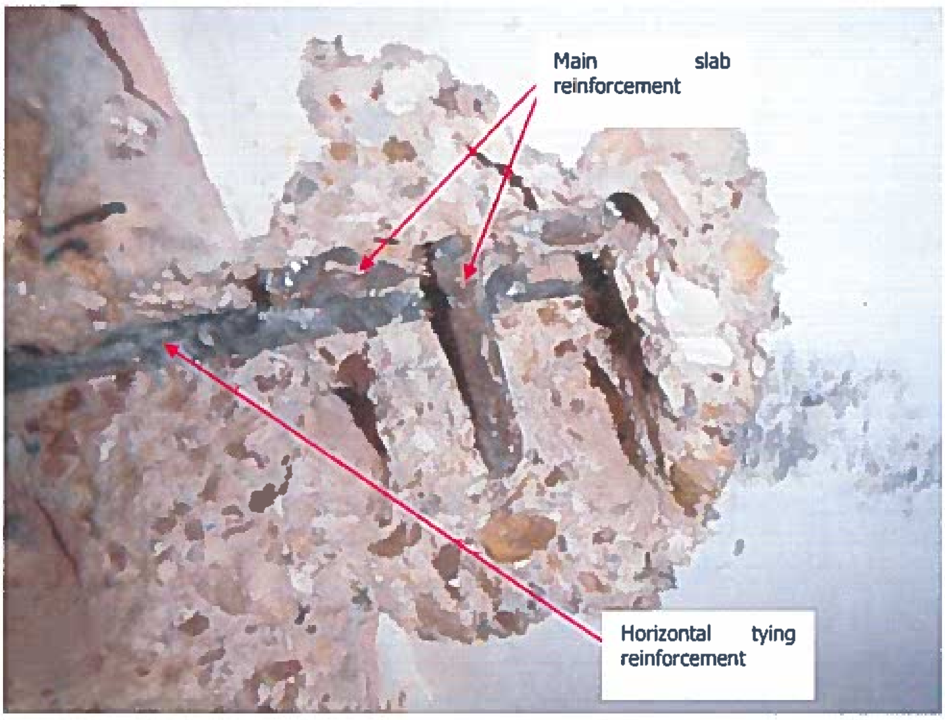
Photograph 19

Breakout U2 tying reinforcement at wall – soffit joint at first floor

Photograph 20

Breakout U3 tying reinforcement at wall – soffit joint at 16th floor



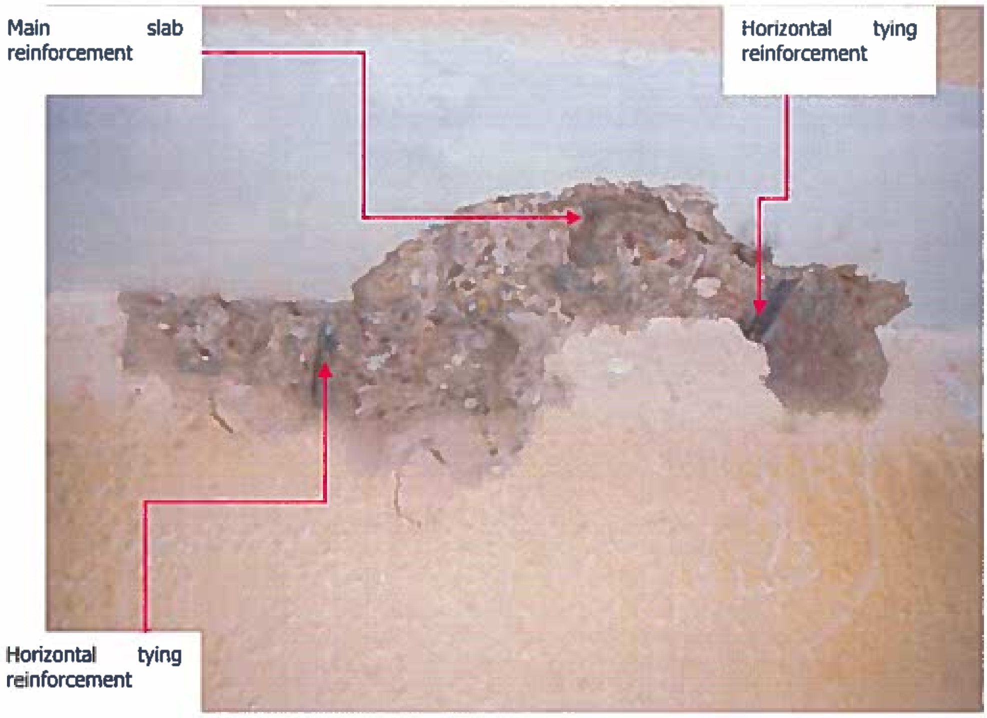


Photograph 21

Breakout U3 tying reinforcement at wall – soffit joint at 16th floor

Photograph 22

Breakout U4 tying reinforcement at wall – soffit joint at 16th floor



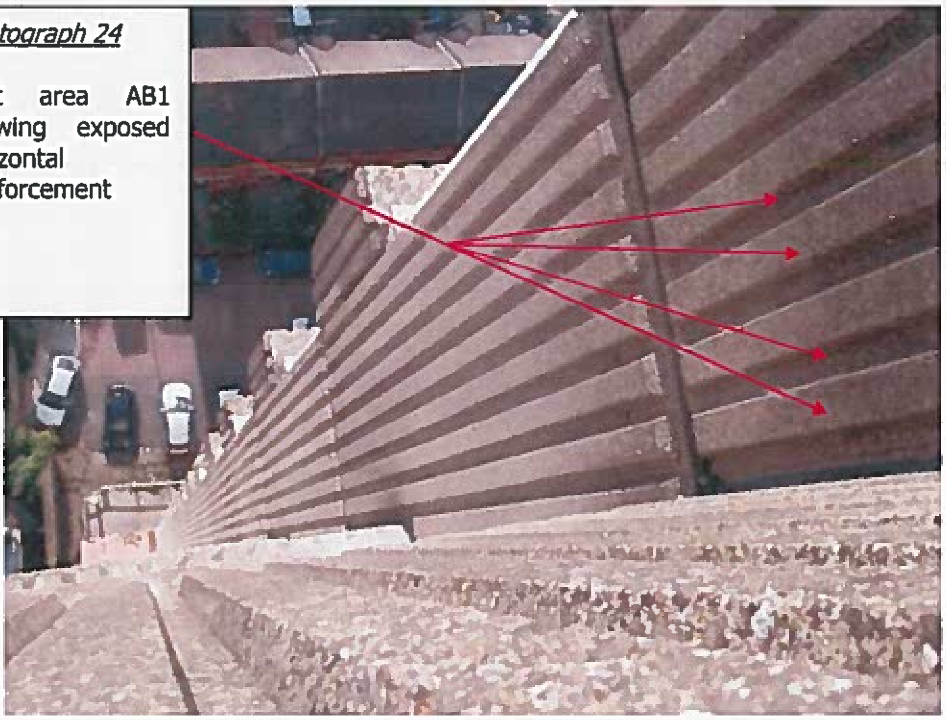
Photograph 23

Breakout U4 at wall – soffit joint at 16th floor

Slab main reinforcement

Photograph 24

Test area AB1 showing exposed horizontal reinforcement



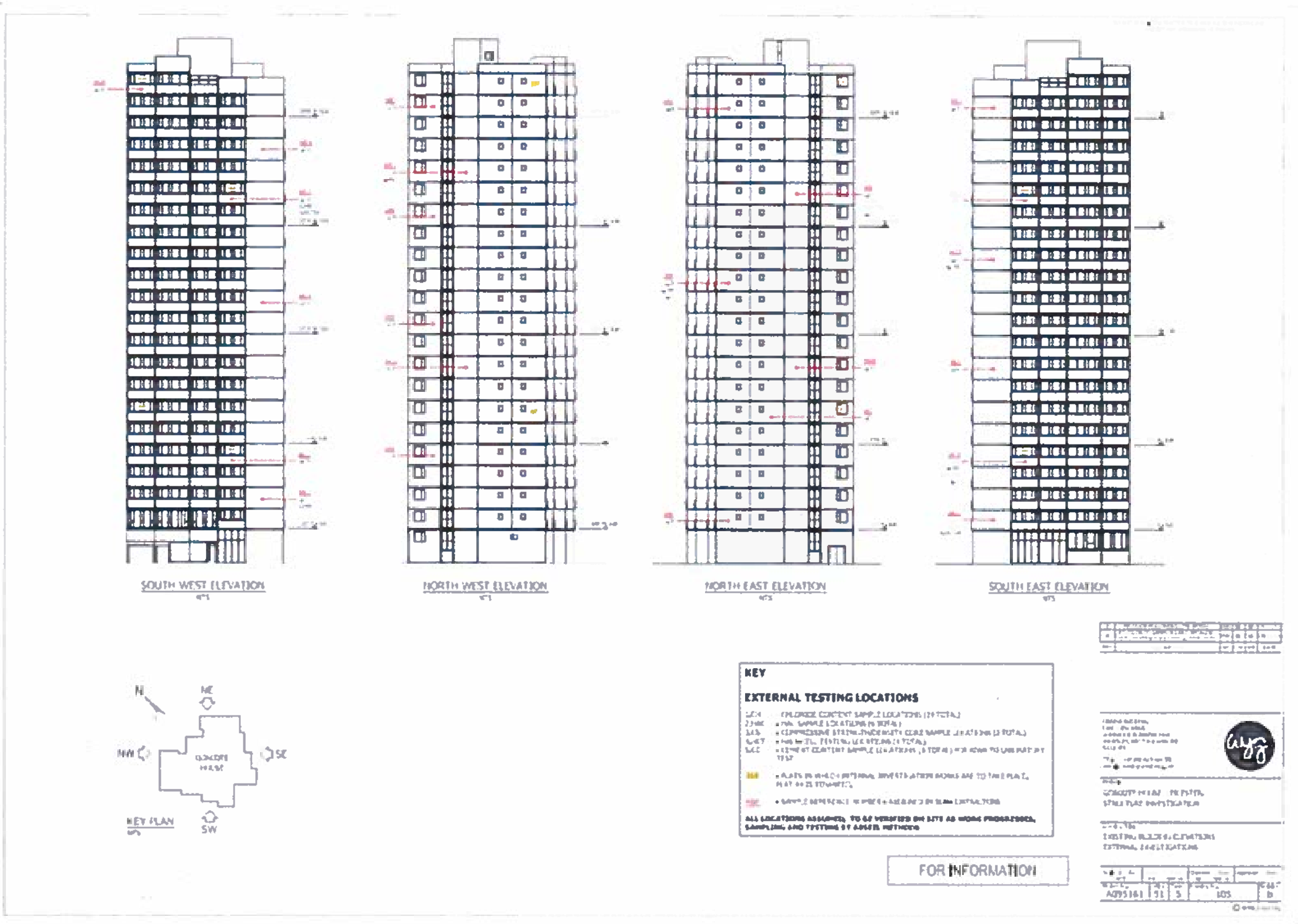
Appendices B – Building Plans Elevations showing Testing Areas

Appendices B1. WYG Drawing A095161-51-S-100B - Existing Building Plans – Internal Investigations


GROUND FLOOR PLAN
4TH FLOOR PLAN
TENANTED FLAT - NO WORKS UNDERTAKEN
16TH FLOOR PLAN
FIRST FLOOR PLAN
6TH FLOOR PLAN
21ST FLOOR PLAN
KEY PLAN
NE
NW
SE
SW
TYPICAL WALL/FLOOR JOINT (DRY PACK TEST)
KEY
INTERNAL TESTING LOCATIONS
FOR INFORMATION

Appendices B2. WYG Drawing A095161-51-S-105B - Existing Building Elevations – External Investigations

READINGS


SOUTH WEST ELEVATION
NORTH WEST ELEVATION
NORTH EAST ELEVATION
SOUTH EAST ELEVATION
N
NE
NW
SE
SW
KEY PLAN
KEY
EXTERNAL TESTING LOCATIONS
FOR INFORMATION