

**CONCRETE STRUCTURE ASSESSMENT REPORT
NEWPORT CENTRAL LIBRARY
NEWPORT**



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1.0 INTRODUCTION

- 1.1 Shear design Ltd have been commissioned by Newport Norse to procure, manage and comment on concrete testing and undertake a visual inspection along with ancillary inspections undertaken by third parties to Newport Central Library, Newport, with the aim of providing the client with suitable remediation strategy recommendations. In addition Shear Design have been commissioned to comment upon the suitability of the existing scaffolding in place to the West Elevation as a safe protection/barrier for any falling debris.
- 1.2 A previous report/suite of surveys was carried out on the building in 2007, report undertaken by Stirling Maynard, Ref AND/PGB/PB/4824. The report and selection of elevations that were marked up were forwarded to ourselves and have been referenced within this report, those referenced are included within [Appendix V](#) – Previous Engineers Survey.
- 1.3 Shear Design Ltd have inspected the building, 21st March 2016. The purpose of this visit was to visually inspect the condition of the structural concrete that forms the structural frame of the building. Photographs were taken during this inspection and are referenced throughout the various sections of this report. Photographs that are referenced are included within [Appendix i](#) – Photographs.
- 1.4 Richard Sharp Specialist Services Ltd were commissioned to undertake concrete tests on Date. The results of these tests were received by Shear Design Ltd on Date and are attached in Appendix iii.
- 1.5 The results of Richard Sharp Specialist Services Ltd concrete tests were reviewed by Shear Design Ltd and compared to guidelines given by BRE Digest 444 part 2, attached in [Appendix iv](#).
- 1.6 A specialist Abseiling survey was commissioned so that to provide drawings and detailed information regarding condition of the existing elevations. These findings are included within [Appendix ii](#) – Abseiling Survey.
- 1.7 A photographic wall survey was undertaken by others, prior to our commission. We have commented upon photos received.
- 1.8 Guidance from the following publications have been utilised in the preparation of this report;
- “Appraisal of Existing Structures – by Institute of Structural Engineers”***
- 1.9 We have not inspected all areas of the building or other parts of the structure which are covered, unexposed or inaccessible and we are therefore unable to report that any such part is free from defect. Foundations were not inspected. We cannot be held responsible for unforeseen conditions which may exist that are covered. Conditions may exist beneath the structure that are not detectable from our visual inspection.
- 1.10 This report is personal to Newport Norse, is confidential and non-assignable unless noted otherwise in the Appointment Documents. Shear Design accepts no liability for losses suffered by any third parties as a result of reliance on this report.

2.0 SHEAR DESIGN INSPECTION

2.1 Shear Design Ltd visited Newport Central Library on Date with the aim of understanding the construction of the building and gaining an insight into the current condition of the building.

2.2 Shear design began the survey by inspecting the internal building space currently used as an active public library ([Photographs 1](#) to [4](#)).

The structural building frame within the public building space is covered in finishes and therefore not exposed. It was not possible to visually inspect the condition of the building materials in these areas however there did not appear to be any evidence of cracking, spalling or staining evidence through the internal building finishes.

The general impression of the internal building space was that the building and its associated finishes showed to be in reasonable condition and free from obvious signs of defect.

2.3 The next area of the building to be inspected was the basement carpark. The structural frame within the car park was not finished and therefore it was possible to confirm the construction as being mostly concrete frame ([Photographs 5](#) to [8](#)) however a hot rolled steel braced bay was also captured ([Photograph 9](#)) along with some masonry wall panels ([Photographs 10](#) to [12](#)).

Apart from some staining to areas of the concrete faces, the concrete within the car park appeared to be in relatively fair condition with no obvious signs of damage in the form of cracking and spalling.

2.4 Once the internal area of the building had been inspected, Shear Design proceeded to commence their walk around survey of the external building. Three of the four building elevations are enclosed within other surrounding building structures therefore Shear Design were unable to inspect these elevations without further specialist equipment.

With the use of a Cherry Picker style elevating platform, Shear Design inspected the West Elevation so that to closely inspect the condition of the external building structure. The north elevation is formed in two main finishes to the concrete frame. The upper portion of the elevation is finished with rough exposed aggregate concrete panels whilst the lower portion is finished with dark masonry/brick slip cladding.

2.5 Upon the initial inspection of the upper West external building elevation, it was clear that the condition of the external concrete was not clear of defect and damage.

[Photograph 13](#) shows patches of spalling to two of the rough aggregate finished panels on the north elevation. Moving along the elevation, further spalling of the same kind, along with extensive staining was observed below the window openings, [Photograph 14](#) and [15](#).

[Photograph 16](#) demonstrates how the entire corner of one of the rough coated panels has broken away from the remaining panel.

Upon inspecting the underside of a window opening further, it was observed that the concrete forming the main body of the concrete wall panels showed evidence of patch spalling and staining ([Photographs 17](#) and [18](#)) whilst the corner of the exposed rough aggregate face of another particular panel has almost all broken away, leaving the main concrete body of the panel almost exposed from the front ([Photograph 19](#)).

- 2.6 Moving down to the lower portion of the West elevation, the signs of damage within the concrete frame and associated finishes continued to be evident.

[Photograph 20](#) shows extensive cracking to the masonry brick slip, whilst moving further down again, a length of brick slip in line with the top of a window opening had moved away from the concrete behind ([Photograph 21](#)).

Possibly the most severe evidence of damage to the external concrete frame was to the concrete lintel/beam over the main window opening in the lower brick slip portion of the west elevation.

[Photographs 22](#) to [25](#) show how large portions of the concrete had completely sprawled away from the main structural elements to the extent where the steel reinforcement within the concrete was becoming exposed ([Photograph 25](#)).

[Photograph 26](#) shows further severe concrete decay with the entire strand of steel reinforcement exposed and almost separated from the main body of the concrete beam behind. A further number of instances of concrete decay and spalling are highlighted on [Photograph 27](#).

- 2.7 In general, Shear Design's reporting's on the condition of the external face of the west elevation would concur with the extensive damages reported by the previous engineering survey of the west elevation carried out by Stirling Maynard & Partners Consulting Engineers 2007, [Appendix v](#).

Due to the similarities between Shear Design's observations of the west elevation and those recorded on the previous engineer's survey findings, it would be fair to assume that the findings on their remaining elevation mark ups are accurate therefore confirming further extensive concrete defect to both the south and east building elevations.

To further confirm this, an abseiling survey specialist were employed to report on the remaining, less accessible elevations.

- 2.8 The scaffolding that is insitu to the lower levels of the West Elevation as we understand been condemned historically as unsafe to access.

The scaffolding boards in some locations are showing signs of severe rotting, and the overall condition of the scaffolding is poor and shows signs of limited maintenance for some time.

3.0 ABSEILING ELEVATION SURVEY

- 3.1 In order to provide a detailed survey of the condition of the remaining building elevations that were obtruded by adjoining buildings, a specialist abseiling team were commissioned to survey the elevations via rope access from the roof of Newport Central Library.

The full survey inclusive of photographs taken by the specialist is included within [Appendix ii](#).

As part of our reporting process, we have detailed a set of building elevation incorporating the comments from the abseiling specialist so that to visually demonstrate the areas of defect to the concrete building frame across each of the building elevations attached within [Appendix vii](#).

- 3.2 The first elevation to be reported on within the cladding inspection was the north elevation, the elevation that the previous engineers mark ups show the lowest extent of defect and damage.

21 no. drops were carried out on the north elevation. The Photographs taken during these drops confirm that the extent of the north elevation to be clad with aluminium panels fixed back to the concrete frame behind. This would explain why the observable defect the concrete is less observable on this elevation, this is not to say that there may be extensive spalling behind the cladding panels.

The reporting's on the north elevation did pick up a vertical fracture through the brickwork beneath the aluminium panels however no other evidence of concrete defect or damage was reported on this elevation.

- 3.3 The next elevation to be surveyed was the east elevation. 13 no. drops were carried out on this elevation, the majority of which resulted in observed damage to the concrete forming both the concrete beams and tiles.

The Photographs and descriptions included within [Appendix ii](#) for the east elevation show the spalling and decay of the concrete structure to be similar to that observed on the west elevation by Shear Design.

A number of tiles are pictured showing the surface at the corners has broken away from the remainder of the tiles whilst the areas of brick slip cladding show evidence cracking.

- 3.4 The third elevation scaled by the abseiling team was the south elevation. 15 no. drops were carried out on this elevation and both the reporting's and Photographs prove the finish of the building to match the west and east elevations with rough aggregate concrete tiles to the upper portion and brick slip cladding to the lower portion.

The Photographs and comments on the south elevation show widespread severe decay to the concrete frame. [Photograph 51](#) within [Appendix ii](#) shows a particularly extreme case of spalling behind the brick slip cladding where the reinforcement within the concrete frame is very much viable.

The remainder of the Photographs taken of the south elevation by the abseiling team show a number of cases of the same damage observed on the west and east elevations, if not more severe in specific locations.

- 3.5 As well as surveying the three elevations that Shear Design were unable to access due to the adjoining buildings, the Abseiling team also carried out 13no. drops to the west elevation. The results of their survey on the west elevation confirmed the findings of both Shear Design's survey and the previous engineers survey, all of the defects picked up by Shear Design were re-photographed and are included within the abseiling report, [Appendix iv](#).

4.0 CONCRETE TESTING REPORT

- 4.1 It is clear from both the walk around survey carried out by Shear Design and the further survey to the remaining building elevations by the specialist abseiling team, that there is extensive decay and damage in the form of spalling and cracking to at least three of the building elevations and therefore the damage is widespread and not specific to one localised area of the building frame.

So that to provide insight to possible root cause and fundamentally understand the correct recommendations for the restoration of the building elevations, Richard Sharp Specialist Services Ltd were commissioned to carry out extensive concrete testing.

- 4.2 Richard Sharp Specialist Services visited the building on the 13th June 2016, and their subsequent reporting's are attached within [Appendix iii](#) – Concrete Testing Report.

The report confirmed the construction of the building being concrete frame with masonry walls within stairwells and reinforced concrete walls to the lift shafts.

- 4.3 The inspection of the internal concrete concurred with the findings of Shear Design in that although the majority of the concrete was obscured with finishes, where exposed, it typically appeared to be in reasonable visual condition.

Chemical testing of the internal concrete showed that elevated chloride levels were identified within the internal concrete, although there was no significant concrete gradient into the concrete, therefore suggesting the chlorides were included within the concrete at the time of construction.

The concrete cover to reinforcement was found to exceed the carbonation depth, concluding the durability of the internal concrete to be intact and that providing the concrete is kept dry, the chloride levels should not induce any premature corrosion and deterioration.

- 4.4 Similar and more extensive testing was also carried out to external concrete to the three building elevations showing evidence of widespread deterioration (west, south and east). Elevated chloride were identified in a number of samples, many of which greater than the 0.4% limit for typical reinforced concrete as suggested in the BRE Digest 444 attached in [Appendix v](#).

The gradient of this chloride was similar to that found within the internal concrete therefore suggesting it was included during the construction stage due to a number of possible reasons.

The report makes comment that the elevated chloride levels allied to localised areas of low concrete cover, therefore likely to have initiated corrosion of the steel reinforcement which has then resulted in further spalling to the concrete.

- 4.5 The report finishes with brief reporting that repairs have been attempted and failed to the deteriorated concrete elements and that future external repairs will need to provide further protection to minimise the possibility of future deterioration occurring.

The report summarises the possible repairs as including an anti-carbonation coating and possibly a corrosion inhibitor.

5.0 ASSESSMENT

- 5.1 Shear Design have been in receipt of a previous engineers structural comments ([Appendix vi](#)) on each of the building elevations to Newport Central Library along with more recent documentation carried out by a specialist abseiling team ([Appendix iv](#)) which has been detailed accordingly onto survey drawings by Shear Design, attached within [Appendix vii](#).

All of the items above demonstrate that there is widespread deterioration to the concrete frame and areas of concrete cladding that form the west, east and south elevations. It is also possible that there could be similar decay to the concrete on the north elevation however, the large proportion of this concrete is hidden beneath non Cementous aluminium cladding panels.

Confirming the above, Shear Design has carried out a walk around inspection of the building, included a more detailed inspection of the west elevation and can concur that widespread deterioration to the external concrete appears to be an issue.

- 5.2 The reporting's of the concrete testing confirm that there are elevated chloride levels across the tested samples both internally and externally. The report concurs with Shear Design's inspection, stating that internally the concrete appears to be in a far more reasonable condition, likely due to the drier environment. This is backed up by the tables provided in the BRE digest 444 that show a 40 year concrete with chloride content of 0.5% (typically the average value shown in the concrete testing) is only moderately at risk of corrosion in a dry environment whilst it is at a high risk in damp environment.

It would be fair to assume that the concrete to the externals of the building is subject to a much damper environment than the internal structure.

- 5.3 None of the survey information received reported on damage to the concrete to the north elevation. Due to the widespread nature of the deterioration to the concrete to the remaining external concrete elevations, unless it has been kept dry, it is unlikely that the concrete beneath the aluminium panels area is free from defect and that said panels are likely to be a visual shield so that it is not observed.

6.0 CONCLUSIONS

- 6.1 The assessment of all of the survey information, both historical and recent, has been reviewed and compared to Shear Design's visual observations. Concrete testing has also been carried out and assessed and therefore the following conclusion can be drawn up regarding the condition and causes of deterioration to the concrete structure at Newport Central Library:-

- The external concrete is subject to widespread deterioration across all three of visible elevations (south, east and west), inclusive of Cementous cladding panels and brick slip/masonry wall panels.
- The root cause of the deterioration appears to be spalling of the concrete caused by corrosion to the steel reinforcement, caused by an elevated chloride content and damp environment.
- The internal concrete is of a much more reasonable condition due to the drier nature of the internal environment.
- It is possible that the concrete on the north elevation behind the aluminium cladding and any other external concrete is also subject to similar deterioration to the remaining elevation although such defect is visually obscured by the cladding.
- From photographic survey provided, it is apparent there has been a systematic remedial wall tie exercise undertaken, this is further supported by pilot/drill holes that are visible within brickwork to the elevations. From the photographs these appear to be of appropriate form and at the required centres as in accordance with Building Regulation requirements.

- Scaffolding to West Elevation is of poor condition and should and in our considered opinion should not be relied upon to provide restraint to any falling debris.
- Though no integral dilapidation has been found within the precast panels, there is a considerable number of panels that have historically and more recently spalled corners etc. This could indicate some poor detailing within the panels.

7.0 RECOMMENDATIONS

7.1 Based on the conclusions within section 6.0 of this report and all of the information included within both the main body of this report and appendices, the following recommendations are applicable so that to provide a suitable repair method to the deteriorated concrete at Newport Central Library:-

- The cladding to the elevations should be locally removed in selected areas so that to provide a representative observation of the concrete behind the cladding on this elevation so that to decide if it should be subject to the treatments outlined in the following points. There is a possibility that this could be undertaken internally to avoid the removal of the panels, but this will be subject to an initial location being opened up and inspected to determine if this is possible. If an internal obtrusive inspection is possible, it should be noted that the opening up works would be noisy and dusty and would impact upon operations on the relevant areas.
- All delaminated concrete should be removed from the elevations using hand tools.
- Brick slip cladding should be removed from the concrete faces to allow the following stages to be completed.
- All exposed reinforcement should be wire brushed and primed. (As per the specification in Appendix vii).
- All spalled concrete to be reinstated to the original profile using a polymer modified repair mortar. (See [Appendix vi](#) for product).
- A new 2mm polymer modified levelling mortar should be applied to all concrete faces. (see [Appendix vi](#) for product).
- Any areas of spalling to precast panels should be reinstated with high strength mortar, finish to match existing. Specialist supplier to be consulted.
- A final anti carbonation coating should be applied to all concrete faces. (As per the specification in [Appendix vii](#)). This would provide a 10-15 year working life, though there would be no warranted design life. A monitoring program would need to be implemented this can be discussed and agreed with product supplier, though we would envisage initial inspection to be 5 years post application, then every other year for two inspections, i.e. 7 and 9 years respectively post application, then yearly until requiring replacement
- Brick slip cladding can be reinstalled on new protected concrete face.
- Cracked masonry panels should be stitched appropriately using a similar product to that specified in [Appendix vii](#).
- Scaffolding should be inspected by Scaffolding supplier and remediated in temporary condition to ensure sufficient integrity to act as restrain to falling debris. Though this should only be a temporary measure until a permanent solution is derived.
- To negate the requirement for scaffolding to protect from falling debris, a permanent solution should be derived. This could be some form of netting, that would be subject to detailed design with planning considerations as deemed necessary by the detailed designer(s).

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APPENDIX

i) PHOTOGRAPHS



PHOTOGRAPH 1



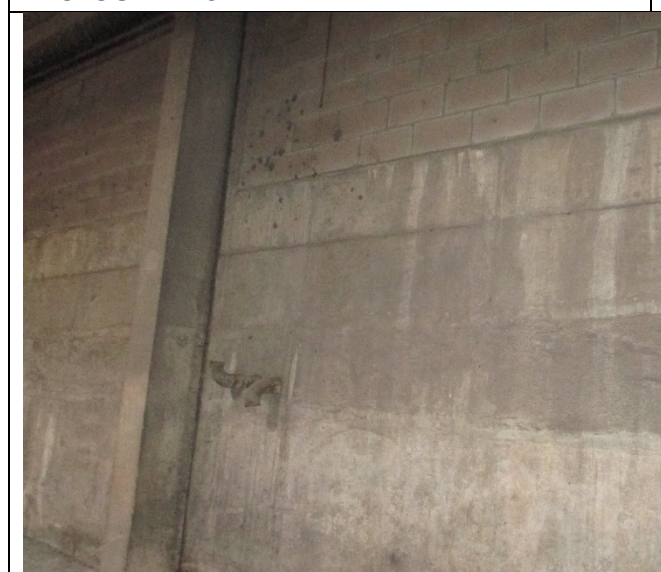
PHOTOGRAPH 2



PHOTOGRAPH 3



PHOTOGRAPH 4



PHOTOGRAPH 5



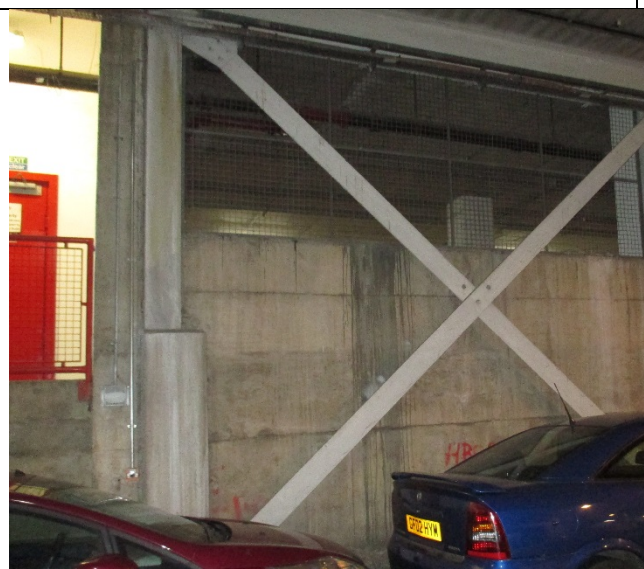
PHOTOGRAPH 6



PHOTOGRAPH 7



PHOTOGRAPH 8



PHOTOGRAPH 9



PHOTOGRAPH 10



PHOTOGRAPH 11



PHOTOGRAPH 12



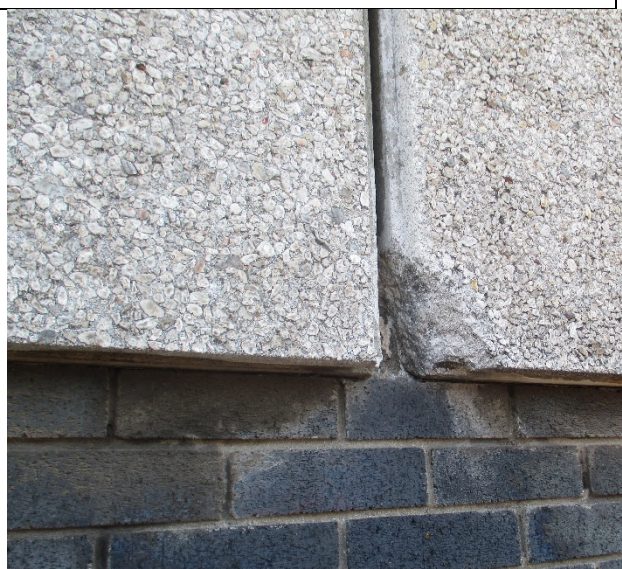
PHOTOGRAPH 13



PHOTOGRAPH 14



PHOTOGRAPH 15



PHOTOGRAPH 16



PHOTOGRAPH 17



PHOTOGRAPH 18



PHOTOGRAPH 19



PHOTOGRAPH 20



PHOTOGRAPH 21



PHOTOGRAPH 22



PHOTOGRAPH 23



PHOTOGRAPH 24



PHOTOGRAPH 25



PHOTOGRAPH 26



PHOTOGRAPH 27

APPENDIX

ii) ABSEILING SURVEY REPORT



CLADDING INSPECTION

Newport Library and Museum

NATIONAL GRID REFERENCE/ NEAREST POST CODE

ST 3132 8789/ NP20 1ED

DATE OF INSPECTION

04-08/04/2016

JOB NUMBER

2381

REPORT REVISION RECORD

REVISION	DATE	ORIGINATOR
0	13/04/16	P. Helm MEng

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ANNEXES

Annex A PHOTOGRAPHS

1 INTRODUCTION

XEIAD has been contracted by NEWPORT NORSE to carry out a Cladding Inspection of Newport Library & Museum.

This report covers the visual and tapping inspection of the external visible areas of the structure. The inspections were carried out using rope access techniques.

SKYLINE ROPE ACCESS LTD has provided personnel and equipment in accordance with the company's operating procedures incorporating relevant health and safety legislation.

ELEVATION OF STRUCTURE



2 INSPECTION NOTES

2.1 East Elevation

Note: cladding panels are labelled 1 to 4 from north to south with each drop between 2&3, rows T for the top row, M for the middle row and B for the bottom row of concrete panels.

2.1.1 Drop 1

Photo 1	Spalling to the north edge of T4 = 50mm H x 15mm W x 5mm D
Photo 2	General feature – air vent top of M1 & M2
Photo 3	Spalling to the south edge of M1 = 50mm H x 15mm W x 10mm D
Photo 4	Spalling to the south edge of M1 = 30mm H x 15mm W x 10mm D
Photo 5	Spalling to the underside of T1 & T2 = 10mm H x 150mm W x 100mm D
Photo 6	General view of the aluminium cladding just north of drop 1
Photo 7	General feature – light fitted between B2 & B3

2.1.2 Drop 2

Photo 8	Spalling to the top north half of M4 = 2.2m H x 320mm W x 5-25mm D
Photo 9	Imminent spalling to the bottom north half of M4 = 2.5m H x 320mm W
Photo 10	Spalling to the bottom south corner of M3 = 30mm H x 80mm W x 60mm D
Photo 11	General view of drop 2
Photo 12	General view of drop 2

2.1.3 Drop 3

Photo 13	Spalling to the north edge of T2 = 500mm H x 50mm W x 50mm D (exposed reinforcement subsequently corroded)
Photo 14	Spalling to the bottom edge of M4 = 30mm H x 100mm W x 10mm D
Photo 15	General view of drop 3

2.1.4 Drop 4

Photo 16	General view of drop 4
Photo 17	General view of drop 4

2.1.5 Drop 5

Photo 18	General view of drop 5
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2.1.6 Drop 6

Photo 19	General view of drop 6
Photo 20	General view of drop 6

2.1.7 Drop 7

Photo 21	Spalling to the top north corner of M3 = 90mm H x 70mm W x 5-30mm D
Photo 22	General feature – light fitted between B2 & B3
Photo 23	General view of drop 7
Photo 24	General view of drop 7

2.1.8 Drop 8

Photo 25	Spalling to the top south corner of M3 = 30mm H x 30mm W x 20mm D
Photo 26	Spalling to the top south corner of M1 = 30mm H x 20mm W x 20mm D
Photo 27	General view of drop 8
Photo 28	General view of drop 8

2.1.9 Drop 9

Photo 29	General view of drop 9
Photo 30	General view of drop 9

2.1.10 Drop 10

Photo 31	Spalling to the south edge of T1 = 60mm H x 20mm W x 20mm D
Photo 32	General view of drop 10

2.1.11 Drop 11

Photo 33	General feature – stairwell window top of B4 (continues into drop 12)
Photo 34	General feature – stairwell window top of M4 (continues into drop 12)
Photo 35	Typical paint loss to window elements
Photo 36	General view of drop 11
Photo 37	General view of brickwork cladding beneath drop 11 – appears to have been recently repointed

2.1.12 Drop 12

Photo 38	Spalling to the bottom north corner of T1 = 70mm H x 70mm W x 70mm D
Photo 39	General feature – stairwell window top of B1 & B2 (continues from drop 11)
Photo 40	General feature – stairwell window top of M1 & M2 (continues from drop 11)
Photo 41	General view of drop 12
Photo 42	General view of brickwork cladding beneath drop 12

2.1.13 Drop 13

Note: only 3No vertical lines of panels (including corner panel) present in this drop.

Photo 43	Spalling to the top of M3 = 580mm H x 320mm W x 20mm D (extends into drop 1 – South Elevation)
Photo 44	Vertical fracture to the north edge of T3 = 2.5m H x HL-5mm wide
Photo 45	General feature – light fitted between B1 & B2
Photo 46	General view of drop 13
Photo 47	General view of brickwork cladding beneath drop 13 showing vertical fracture, located 900mm in from the south edge of east elevation = 2.2m H x HL-2mm W

2.2 South Elevation

Note: cladding panels are labelled 1 to 4 from east to west with each drop between 2&3, rows T for the top row, M for the middle row and B for the bottom row of concrete panels. There are 4 rows of windows spanning across south elevation between drops 2 and 13 and are labelled window rows 1 to 4 from top to bottom. Elements obstructed by scaffold on the west lower section of south elevation were not inspected.

2.2.1 Drop 1

Photo 48	Vertical fracture to the west edge of T1 = 2m H x HL-3mm wide
Photo 49	Spalling to the top half of M1 = 920mm H x 320mm W x 25mm D
Photo 50	Spalling to the west side of M1 = 820mm H x 90mm W x 25mm D
Photo 51	Line of missing brickwork located mid height in line with window row 3, 9No bricks missing (2.8m L). Associated exposed reinforcement has subsequently corroded.
Photo 52	Vertical stepped fracture, steps up from the east end of the line of missing brickwork = 1m H x HL-2mm. It appears that the fracture has been previously repaired but has now begun to separate again. There is associated displacement along the length of the fracture with brickwork protruding latterly 10-30mm.
Photo 53	Vertical stepped fracture, steps down from the east end of the line of missing brickwork = 1.2m H x HL-1mm.
Photo 54	Vertical stepped fracture, steps down from the centre of the line of missing brickwork = 1.6m H x HL-2mm. Half way down the fracture there is additional horizontal fracture = 1.2m L x HL-1mm.

No Photo	Area of brickwork cladding beneath the line of missing bricks was bulging out by up to 35mm with associated hollowness affecting approximately 2m ² .
Photo 55	Line of missing brickwork located in line with window row 4 lintel, 8No bricks missing. Area of missing brickwork has been filled with mortar.
Photo 56	Areas of calcite staining to brickwork cladding beneath concrete cladding = approximately 3m ² .
Photo 57	Vertical fracture to the base of brickwork cladding beneath B2 = 670mm H x 2mm W, associated spalling mid height of the fracture = 70mm H x 40mm W x 15mm D
Photo 58	General view looking up

2.2.2 Drop 2

Photo 59	Spalling to the bottom east corner of T2 = 70mm H x 50mm W x 40mm D
Photo 60	Spalling to the bottom east corner of T2 = 30mm H x 20mm W x 5-10mm D
Photo 61	Vertical fracture through glass, low levels of window row 1, beneath T4 = 340mm H x HL W
Photo 62	Vertical separation of brickwork cladding, in line with window row 3 beneath B1 = 1.7m H x 5-35mm wide
Photo 63	Vertical stepped fracture through brickwork, located in line with the lower half of window row 3 = 700mm H x 5 – 25mm
Photo 64	Vertical separation of brickwork cladding, in line with window row 4 beneath B1 = 1.7m H x HL-4mm wide
Photo 65	Immanent intermittent spalling throughout full length of window row 4 lintel, between drops 2 and 13
Photo 66	Window row 4 cill loose intermittently throughout full length between drops 2 and 13
Photo 67	General view looking up
Photo 68	General view looking up

2.2.3 Drop 3

Photo 69	Spalling to the west edge of T3 = 140mm H x 40mm W x 55mm D (exposed reinforcement subsequently corroded)
Photo 70	Diagonal fracture through glass, top half of window row 1, beneath T1 = 650mm L x HL W
Photo 71	Typical paint loss to window frame elements, affecting all 4 rows of windows between drops 2 and 13
Photo 72	Horizontal fracture through glass, centre of window row 4, beneath B4 = 550mm L x HL W
Photo 73	Vertical fracture through glass, bottom half of window row 4, beneath T3 = 270mm H x HL W
Photo 74	2No vertical fractures through brickwork cladding, extending down from window row 4 cill beneath B3 = 1.8m H (total linear length) x HL-4mm wide
Photo 75	General view looking up

2.2.4 Drop 4

Photo 76	General view looking up
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2.2.5 Drop 5

Photo 77	Spalling to the underside T1 = 10mm H x 70mm W x 100mm D
Photo 78	Vertical fracture through glass, bottom half of window row 1, beneath T1 = 335mm H x HL W
Photo 79	Diagonal fracture through glass, centre of window row 2, beneath M2 & 3 = 700mm L x HL W
Photo 80	General view looking up

2.2.6 Drop 6

Photo 81	Spalling to the underside T1 = 20mm H x 130mm W x 40mm D
Photo 82	Spalling to the bottom east corner of the window row 1 column beneath T1 & 2 = 20mm

	H x 130mm W x 40mm D
Photo 83	General view looking up

2.2.7 Drop 7

Photo 84	Spalling to the bottom west corner of the window row 1 column beneath T3 & 4 = 360mm H x 25mm W x 40mm D
Photo 85	Spalling to the top west corner of the window row 2 column beneath M3 & 4 = 50mm H x 270mm W x 20mm D
Photo 86	Typical intermittent and immediate spalling to window row 4 lintel with exposed reinforcement
Photo 87	General remark – bottom brickwork cladding beneath window row 4 between drop 7 and drop 11 covered by plastic sheeting, purpose of this is unknown, brickwork beneath plastic sheeting not examined
Photo 88	General view looking up

2.2.8 Drop 8

Photo 89	Intermittent spalling along the east edge of T2 = 1m H (total linear height) x 20mm W x 70mm D, exposed reinforcement subsequently corroded
Photo 90	Spalling to the underside of T2 = 40mm H x 50mm W x 70mm D
Photo 91	Horizontal fracture through glass, centre line of window row 1, beneath T3 = 530mm L x HL W
Photo 92	Diagonal fracture through glass, centre line of window row 1, beneath T1 = 160mm L x HL W
Photo 93	Spalling to the underside west corner of M2 = 20mm H x 50mm W x 70mm D
Photo 94	General view looking up

2.2.9 Drop 9

Photo 95	Diagonal fracture through glass, top of window row 2, beneath M3 = 200mm L x HL W
Photo 96	Missing facing tiles to the bottom half of the window row 2 column beneath M1 & 2 = 700mm H x 610mm W x 20mm D
Photo 97	Typical displacement and loose window row 4 cill (defect extend intermittently full span of window row 4 between drops 2 and 13)
Photo 98	General view looking up

2.2.10 Drop 10

Photo 99	Spalling to the east edge of T2 = 40mm H x 50mm W x 70mm D
Photo 100	Vertical fracture through glass, bottom half of window row 1, beneath T2 = 400mm L x HL W
Photo 101	Vertical fracture through glass, bottom half of window row 4, beneath B2 = 220mm L x HL W
Photo 102	General view looking up

2.2.11 Drop 11

Photo 103	General remark – 250mm diameter air vent in glass, window row 2 beneath M3
Photo 104	General view looking up

2.2.12 Drop 12

Photo 105	General feature – external duct fitting fixed to the column in window row 1 beneath T1 and 2
Photo 106	General feature – external duct fitting fixed to the column in window row 3 beneath B1 and 2
Photo 107	Sporadic spalling to the east edge of column in window row 4 beneath B1 and 2 <0.1m ² x 10mm D
Photo 108	General view looking up
No Photo	General Note – all elements of the south elevation beneath the line of window row 4 cill,

	between drops 12 and 15 were obstructed by scaffold and therefore not examined.
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2.2.13 Drop 13

Photo 109	Spalling to the top of west edge of T3 = 130mm H x 15mm W x 20mm D
Photo 110	Spalling to the west edge of M3 = 440mm H x 30mm W x 15mm D
Photo 111	Spalling to the west edge of M3 = 1.1m H x 60mm W x 80mm D
Photo 112	Spalling to the bottom east corner of M4 = 40mm H x 50mm W x 15mm D
Photo 113	Spalling to the top of west edge of B3 = 760mm H x 70mm W x 80mm D
Photo 114	Imminent spalling to the top of east edge of B3 = 550mm H x 50mm W
Photo 115	Spalling to the bottom of west edge of B3 = 1m H x 55mm W x 90mm D
Photo 116	Vertical stepped fracture through brickwork beneath window row 3 beneath B2 = 1m H x HL-2mm W
Photo 117	Vertical fracture through brickwork beneath B3 = 450m H x HL-2mm W
Photo 118	Line of missing brickwork located in line with window row 4 lintel, 4No bricks missing. Area of missing brickwork has exposed reinforcement.
Photo 119	Vertical fracture extending down from the area of missing brickwork beneath B3 = 2-3mm wide x undetermined length (extends beyond the scaffold)
Photo 120	1No missing brick beneath B4
Photo 121	General view looking up

2.2.14 Drop 14

Note: there are no panels M3 & 4 and B3 & 4 at this drop, instead 3No stairwell windows are present separated by brickwork cladding. For labelling windows are referred to as window 1-4 from top to bottom.

Photo 122	Separation fracture to the east side of window 1 = 1.1m H x 3-4mm W
Photo 123	Separation fracture to the west side of window 1 = 1.1m H x 1-2mm W
Photo 124	Separation fracture to the east side of window 2 = 1.1m H x 2-5mm W
Photo 125	Separation fracture to the west side of window 2 = 1.1m H x HL-2mm W
Photo 126	Loss of joint material to the west edge of T2 & M2, void = 50mm H x 100mm W x 100mm D
Photo 127	Horizontal fracture to window 2 lintel = 400mm L x HL-1mm W
Photo 128	Separation fracture to the east side (top half) of window 3 = 500mm H x 1mm W
Photo 129	Separation fracture to the west side of window 3 = 1.1m H x HL(bottom)-10mm(top) W
Photo 130	Spalling to the window 3 lintel = 50mm H x 1m W x 40mm D (exposed reinforcement)
Photo 131	Vertical stepped fracture through brickwork beneath B1 & 2 = 1.6m H x 1-5mm W
Photo 132	Separation fracture to the west side of window 4 = 1.1m H x 5mm W(bottom)-20mm W(top)
Photo 133	Spalling to the window 4 lintel = 50mm H x 1.5m W x 180mm D (exposed reinforcement)
Photo 134	General view looking up

2.2.15 Drop 15

Note: only 2No vertical lines of panels (including corner panel) present in this drop.

Photo 135	Spalling to the west edge of M1 = 670mm H x 70mm W x 80mm D
Photo 136	Spalling to the west edge of M1 = 80mm H x 20mm W x 20mm D
Photo 137	Immanent spalling to the west edge of M1 = 400mm H x 20mm W
Photo 138	Spalling to the brickwork beneath B1 = 70mm H x 880mm W x 20mm D
Photo 139	Vertical stepped fracture through brickwork beneath B1 = 1.25m H x HL-2mm W
Photo 140	General view looking up

2.3 East Elevation – Brickwork Cladding Beneath Overhang

Note: Inspection findings of brickwork east elevation beneath concrete cladding overhang between drops 1-11. Access gained through the fire escape doors located in the south east corner of the building.

Photo 141	General view of the south return
Photo 142	General view of the east elevation brickwork cladding beneath overhang – looking north
Photo 143	General view of the east elevation brickwork cladding beneath overhang – looking south

2.3.1 Bottom Half – from 2m down to the floor level

Photo 144	Spalling to the east edge of the south return, top of access door = 140mm H x 40mm W x 20mm D
Photo 145	Poor concrete mix at the mitre between south return and east elevation
Photo 146	Sporadic areas of honeycombing throughout concrete section separating red and blue bricks.
Photo 147	Sporadic areas of spalling throughout the line of concrete <0.5m ² x 10-30mm D
Photo 148	Spalling to the concrete section, located 20m from the south return, 2m above floor level = 70mm H x 180mm W x 30mm D
Photo 149	Vertical fracture through concrete section, located 19.5m from the south return, 1.8m above floor level = 360mm H x HL-2mm W
Photo 150	Poor concrete mix pillar located 27.9m from south return
Photo 151	Sporadic debris on floor level
Photo 152	Sporadic debris on floor level

2.3.2 Top Half – from 2m up to the underside of overhang

Photo 153	Typical joint loss throughout mid height of brickwork cladding, affecting <0.5m ² in total to a maximum depth of 20mm
Photo 154	Spalling to brickwork located 2.5m from the south return, 2.3m above floor level = 60mm H x 100mm W x 10mm D
Photo 155	General view of render finish section top of brickwork and just beneath the overhang
Photo 156	General feature – air vent located in render finish 9.9m from the south return, 4.3m above floor level
Photo 157	Vertical fracture through render section, located 9.9m from the south return = 200mm H x 2mm W
Photo 158	Vertical fracture through render section, located 10.7m from the south return = 1m H x HL-3mm W
Photo 159	Vertical fracture through render section, located 12.8m from the south return = 1m H x HL-1mm W
Photo 160	Vertical fracture through render section, located 16.5m from the south return = 1m H x HL-1mm W
Photo 161	3No fractures through render section, located 26.5m from the south return = 2m total linear length x HL-1mm W
No Photo	General feature – air vent located in render finish 27.2m from the south return, 4.3m above floor level
Photo 162	Vertical stepped fracture through brickwork, located 27.6m from the south return, 2.2m above ground level = 300mm H x 5mm W
Photo 163	3No fractures through render section, located 28.2m from the south return = 3.5m total linear length x HL-1mm W
Photo 164	Missing brickwork to the south corner, located 2.9m above floor level = 70mm H x 220mm W x 100mm D
Photo 165	Typical sporadic spalling throughout <0.1m ² in total to a maximum depth of 20mm

2.4 West Elevation

Note: cladding panels are labelled 1 to 4 from east to west with each drop between 2&3, rows T for the top row, M for the middle row and B for the bottom row of concrete panels. There are 4 rows of windows spanning across west elevation between drops 3 and 11 and are labelled window rows 1 to 4 from top to bottom. Elements obstructed by scaffold on the lower section of west elevation were not inspected.

2.4.1 Drop 1

Photo 166	Spalling to the west face of T1 = 2.5m H x 330mm W x 10-30mm D
Photo 167	Vertical stepped fracture through brickwork beneath B3 = 1.5m H x 1-5mm W
Photo 168	3No loose bricks, located 1.5m beneath B2
Photo 169	Vertical stepped fracture through brickwork beneath B1 = 4.2m H x 1-10mm W
Photo 170	Line of missing brickwork, starts approximately 3m beneath B2 and extends into drops 2 & 3 and into window row 4 lintel. 22No bricks missing
Photo 171	Vertical stepped fracture through brickwork, starts 3m beneath B3 and extends down = 1.7m H x 1-5mm W

Photo 172	Typical sporadic joint loss, mainly to lower levels of brickwork in line with window row 4 between drops 1-3 <0.5m ² x30mm D
Photo 173	General view looking up

2.4.2 Drop 2

Photo 174	Spalling to the south edge of T4 = 100mm H x 10mm W x 40mm D
Photo 175	2No loose bricks, located 1.5m beneath B3 & 4
Photo 176	Line of joint loss to brickwork 1.5m beneath B1-4, extends into drop 1 and 3 <1m ² x30mm D
Photo 177	General view looking up

2.4.3 Drop 3

Photo 178	Spalling to the bottom half of the infill between T1 and 2 = 1.1m H x 30mm W x 90mm D
Photo 179	Spalling to the bottom of the window row 2 column beneath M3 = 270mm H x 340mm W x 90mm D
Photo 180	Vertical stepped fracture through brickwork, starts 900mm beneath B2 and extends into window row 3 cill = 1.8m H x 5-10mm W
Photo 181	Immanent intermittent spalling throughout full length of window row 4 lintel, between drops 3 and 11
Photo 182	General view looking up

2.4.4 Drop 4

Photo 183	Spalling to the top north corner of T2 = 120mm H x 200mm W x 20mm D
Photo 184	Spalling to the bottom south corner of T2 = 1.25m H x 250mm W x 30mm D
Photo 185	General view looking up

2.4.5 Drop 5

Photo 186	2No Vertical stepped fracture to window row 2 column beneath M1 and 2 = 1.15m H x HL-5mm W
Photo 187	Spalling to window row 2 column bottom south corner beneath M1 and 2 = 140mm H x 30mm W x 20mm
Photo 188	Spalling to window row 2 column bottom north corner beneath M1 and 2 = 70mm H x 30mm W x 5mm
Photo 189	Spalling to window row 3 column bottom south corner beneath B1 and 2 = 170mm H x 30mm W x 30mm
Photo 190	Spalling to brickwork just above window row 4 lintel beneath B4 = 60mm H x 50mm W x 5mm
Photo 191	General view looking up

2.4.6 Drop 6

Photo 192	Typical paint loss to window frame elements affecting all 4 rows of windows between drops 3 and 11
Photo 193	General view looking up

2.4.7 Drop 7

Photo 194	General feature – light bracket fitted to T2 and 3
Photo 195	Spalling to window row 3 column bottom south corner beneath B1 and 2 = 240mm H x 30mm W x 10mm
Photo 196	Spalling to window row 3 column bottom north corner beneath B1 and 2 = 240mm H x 30mm W x 10mm
Photo 197	Diagonal stepped fracture to window row 2 column beneath B1 and 2 = 600mm L x HL-2mm W
Photo 198	Typical spalling to window row 4 lintel – area with exposed reinforcement (this defect extends intermittently full length of the lintel between drops 3 and 11)
Photo 199	General view looking up

2.4.8 Drop 8

Photo 200	Spalling to the north edge of T1 = 320mm H x 30mm W x 90mm D
Photo 201	General view looking up

2.4.9 Drop 9

Photo 202	Spalling to window row 2 column bottom south corner beneath M1 and 2 = 70mm H x 30mm W x 10mm
Photo 203	Spalling to window row 2 column bottom north corner beneath M1 and 2 = 220mm H x 60mm W x 50mm
Photo 204	General view looking up

2.4.10 Drop 10

Photo 205	Spalling between M3 and 4 beneath window row 2 = 20mm H x 40mm W x 2mm, exposed reinforcement
Photo 206	General view looking up

2.4.11 Drop 11

Photo 207	Spalling to the top north corner of window row 1 column beneath T1 = 70mm H x 50mm W x 10mm d
Photo 208	Spalling to the bottom north corner of window row 1 column beneath T1 = 150mm H x 140mm W x 30mm d
Photo 209	Line of joint loss to brickwork 1.35m beneath B1 and extends into drop 12 and 13 <1m ² x30mm D
Photo 210	1No missing brick, located 1.35m beneath B1
Photo 211	Line of missing brickwork 1.35m beneath B1 and extends into drop 12, 17No bricks missing
Photo 212	Line of joint loss to brickwork 3.7m beneath B1 and extends into drop 12 and 13
Photo 213-215	Vertical stepped fracture extending up and to the north from the cill edge of window row 4 beneath B1. The fracture extends into the line of missing brickwork at 1.35m beneath the bottom edge of concrete cladding at drop 12. Total linear length of the fracture = 4.5m and is HL-3mm wide
Photo 216	General view looking up

2.4.12 Drop 12

Photo 217	Spalling to the bottom north corner of T1 = 30mm H x 100mm W x 50mm D
Photo 218	Spalling to the top south corner of T3 = 500mm H x 450mm W x 40mm D
Photo 219	Spalling to the north corner of B3 = 150mm H x 120mm W x 90mm D
Photo 220	Vertical fracture through brickwork located 4m beneath B3 = 500mm H x 1-2mm W
Photo 221	General view looking up

2.4.13 Drop 13

Note: Two lines of aluminium panels are located on the north end of the west elevation

Photo 222	General feature – window on top of M3 and 4
Photo 223	General feature – window on top of B3 and 4
Photo 224	2No spall repairs to top of B1 and 2, each = 100mm H x 100mm W
Photo 225	Spall repair to south of B1= 230mm H x 150mm W
Photo 226	2No missing bricks to brickwork cladding beneath B1 and B4
Photo 227	Vertical fracture through brickwork started 3m beneath B2 and 3 and extended downwards 1.65m, open 1-3mm
Photo 228	General view looking up

2.5 North Elevation

Note: photo survey of aluminium panels. Drops labelled from west to east

2.5.1 Drop 1

Photo 229	General view looking up
Photo 230	Brickwork cladding at 1 st floor level
Photo 231	West elevation of 1 st floor level extension

2.5.2 Drop 2

Photo 232	General view looking up
Photo 233	General view looking up

2.5.3 Drop 3

Photo 234	General view looking up
Photo 235	General view looking up
Photo 236	General view looking down and to the east

2.5.4 Drop 4

Photo 237	General view looking up
Photo 238	General view looking up
Photo 239	General view looking up
Photo 240	West elevation of 2 nd and 3 rd floor level extension
Photo 241	West elevation of 2 nd and 3 rd floor level extension

2.5.5 Drop 5

Photo 242	General view looking up
Photo 243	General view looking up
Photo 244	General view looking up

2.5.6 Drop 6

Photo 245	General view looking up
Photo 246	General view looking up
Photo 247	General view looking up

2.5.7 Drop 7

Photo 248	General view looking up
Photo 249	General view looking up
Photo 250	General view looking up

2.5.8 Drop 8

Photo 251	General view looking up
Photo 252	General view looking up
Photo 253	General view looking up

2.5.9 Drop 9

Photo 254	General view looking up
Photo 255	General view looking up
Photo 256	General view looking up

2.5.10 Drop 10

Photo 257	General view looking up
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Photo 258	East elevation of 2 nd and 3 rd floor level extension
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2.5.11 Drop 11

Photo 259	General view looking up
Photo 260	East elevation of 1 st floor level extension

2.5.12 Drop 12

Photo 261	General view looking up
Photo 262	Loose letter “E” located on a 3 rd panel down
Photo 263	Vertical fracture through brickwork beneath aluminium panels = 680mm x HL-2mm

2.5.13 Drop 13

Photo 264	General view looking up
Photo 265	Loose letter “M” located on a 4 th panel down
Photo 266	Loose letter “G” located on a 3 rd panel down
Photo 267	Cladding brickwork repair beneath aluminium panels = 740mm H x 1.8m W. 200mm diameter metallic pipe located in the centre of repair patch

2.5.14 Drop 14

Photo 268	General view looking up
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2.5.15 Drop 15

Photo 269	General view looking up
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2.5.16 Drop 16

Photo 270	General view looking up
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2.5.17 Drop 17

Photo 271	General view looking up
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2.5.18 Drop 18

Photo 272	General view looking up
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2.5.19 Drop 19

Photo 273	General view looking up
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2.5.20 Drop 20

Photo 274	General view looking up
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2.5.21 Drop 21

Photo 275	General view looking up
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ANNEX A
PHOTOGRAPHS



Photo 1. Drop 1 – East Elevation



Photo 2. Drop 1 – East Elevation



Photo 3. Drop 1 – East Elevation



Photo 4. Drop 1 – East Elevation



Photo 5. Drop 1 – East Elevation



Photo 6. Drop 1 – East Elevation



Photo 7. Drop 1 – East Elevation



Photo 8. Drop 2 – East Elevation



Photo 9. Drop 2 – East Elevation



Photo 10. Drop 2 – East Elevation



Photo 11. Drop 2 – East Elevation



Photo 12. Drop 2 – East Elevation



Photo 13. Drop 3 – East Elevation



Photo 14. Drop 3 – East Elevation



Photo 15. Drop 3 – East Elevation



Photo 16. Drop 4 – East Elevation



Photo 17. Drop 4 – East Elevation



Photo 18. Drop 5 – East Elevation



Photo 19. Drop 6 – East Elevation



Photo 20. Drop 6 – East Elevation



Photo 21. Drop 7 – East Elevation



Photo 22. Drop 7 – East Elevation



Photo 23. Drop 7 – East Elevation



Photo 24. Drop 7 – East Elevation



Photo 25. Drop 8 – East Elevation



Photo 26. Drop 8 – East Elevation

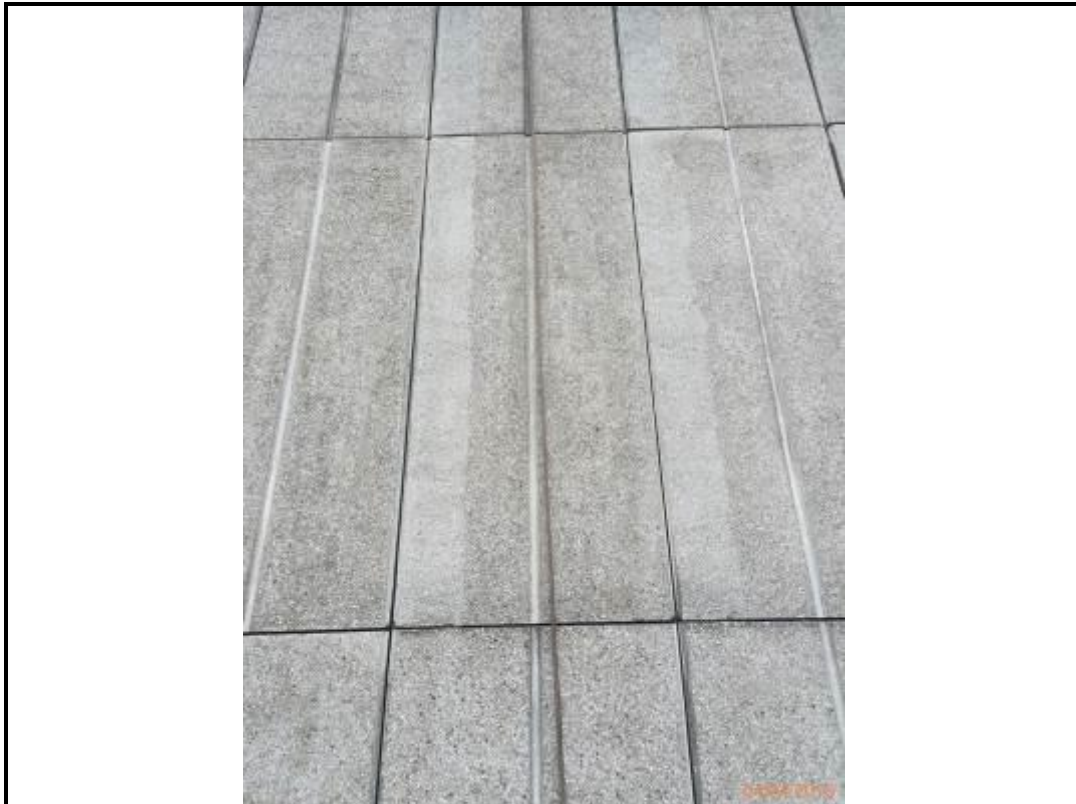


Photo 27. Drop 8 – East Elevation

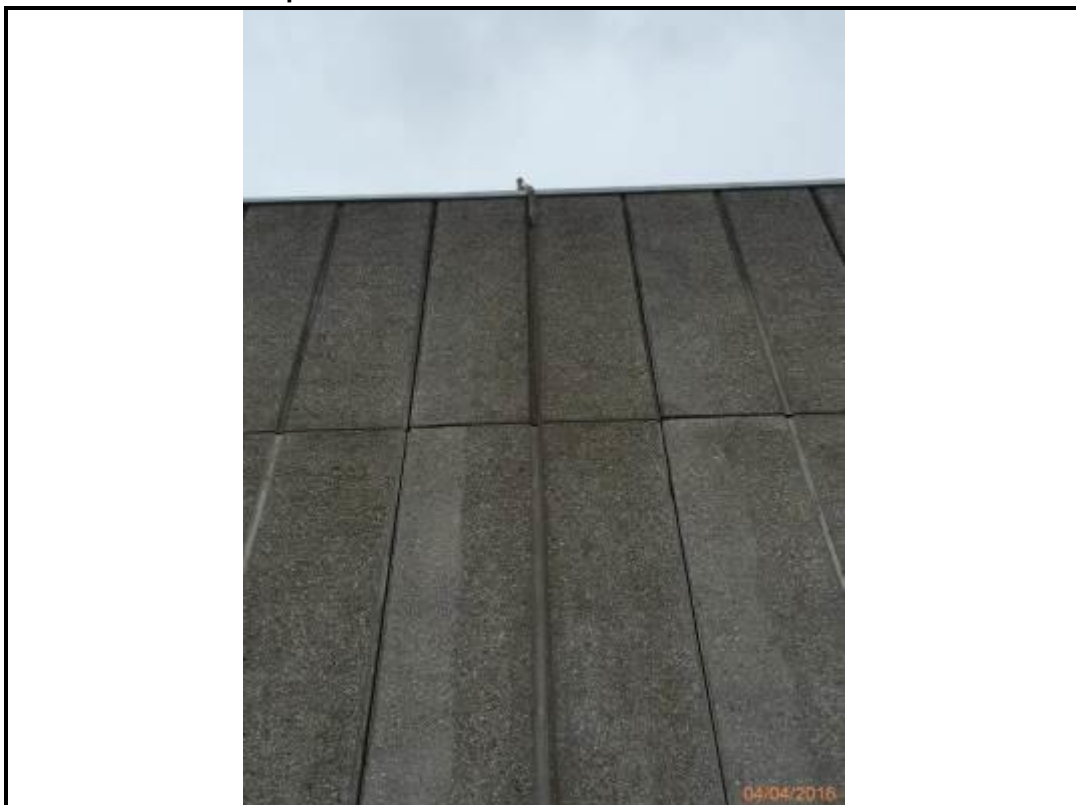


Photo 28. Drop 8 – East Elevation



Photo 29. Drop 9 – East Elevation



Photo 30. Drop 9 – East Elevation



Photo 31. Drop 10 – East Elevation



Photo 32. Drop 10 – East Elevation



Photo 33. Drop 11 – East Elevation



Photo 34. Drop 11 – East Elevation



Photo 35. Drop 11 – East Elevation



Photo 36. Drop 11 – East Elevation



Photo 37. Drop 11 – East Elevation



Photo 38. Drop 12 – East Elevation



Photo 39. Drop 12 – East Elevation



Photo 40. Drop 12 – East Elevation



Photo 41. Drop 12 – East Elevation



Photo 42. Drop 12 – East Elevation



Photo 43. Drop 13 – East Elevation



Photo 44. Drop 13 – East Elevation



Photo 45. Drop 13 – East Elevation



Photo 46. Drop 13 – East Elevation



Photo 47. Drop 13 – East Elevation

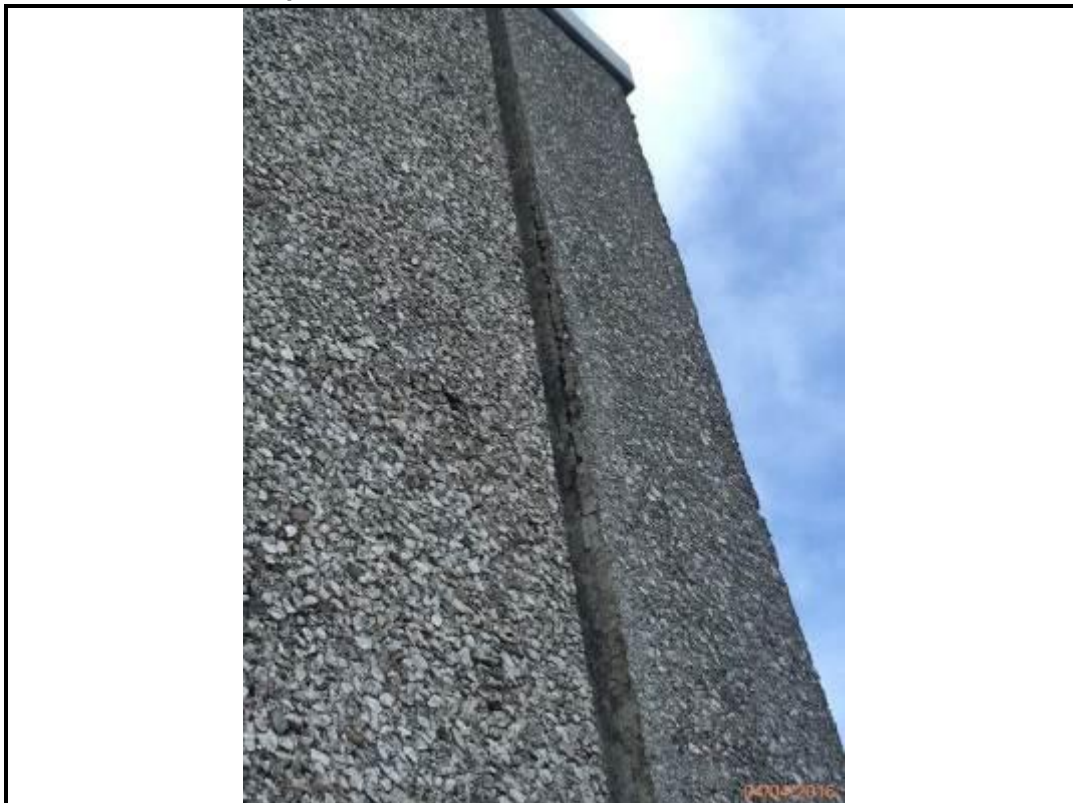


Photo 48. Drop 1 – South Elevation

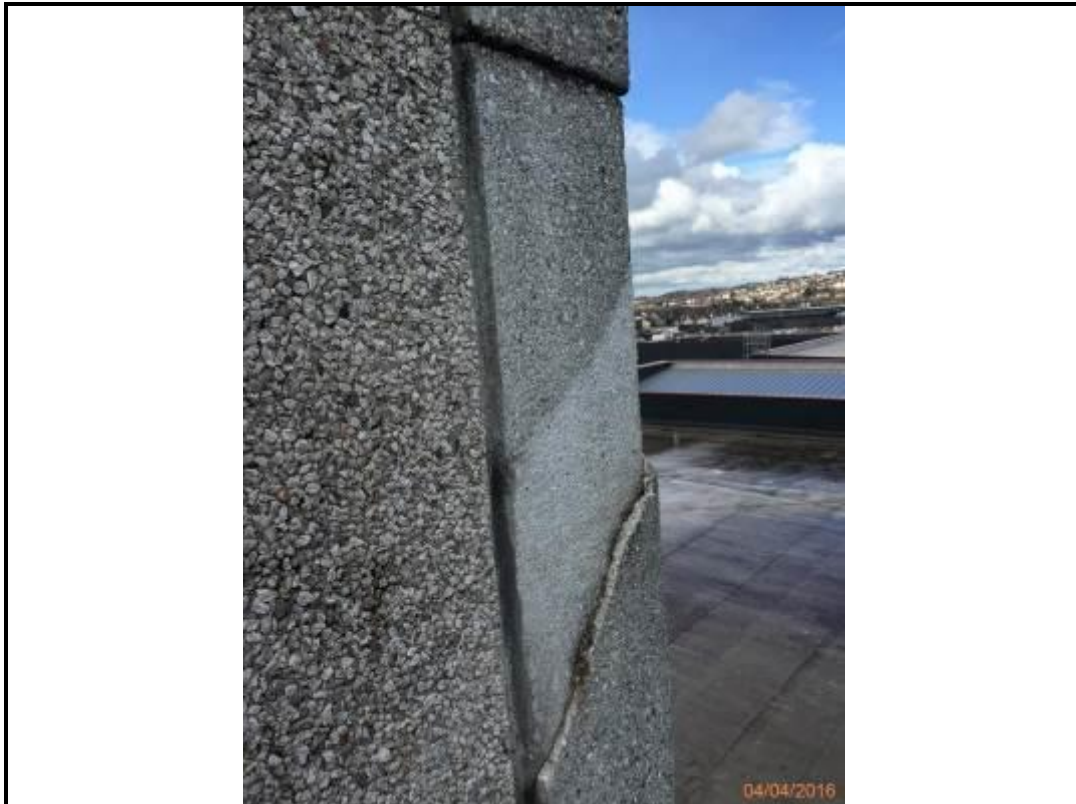


Photo 49. Drop 1 – South Elevation



Photo 50. Drop 1 – South Elevation



Photo 51. Drop 1 – South Elevation

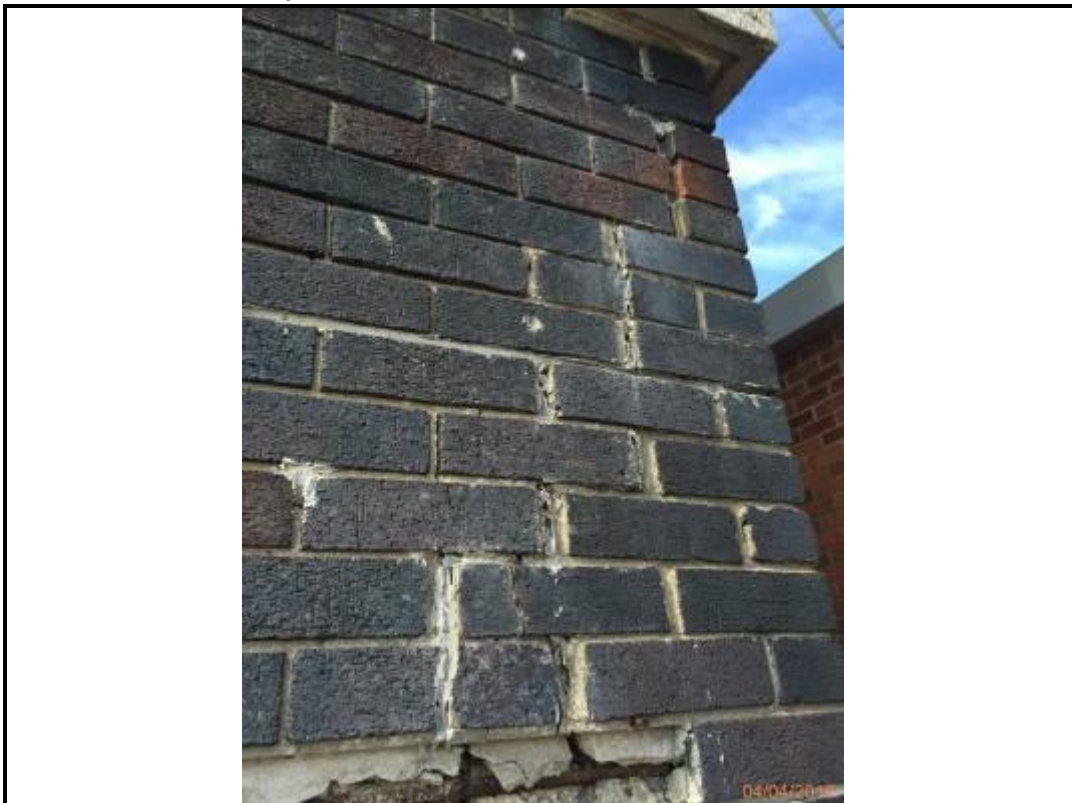


Photo 52. Drop 1 – South Elevation

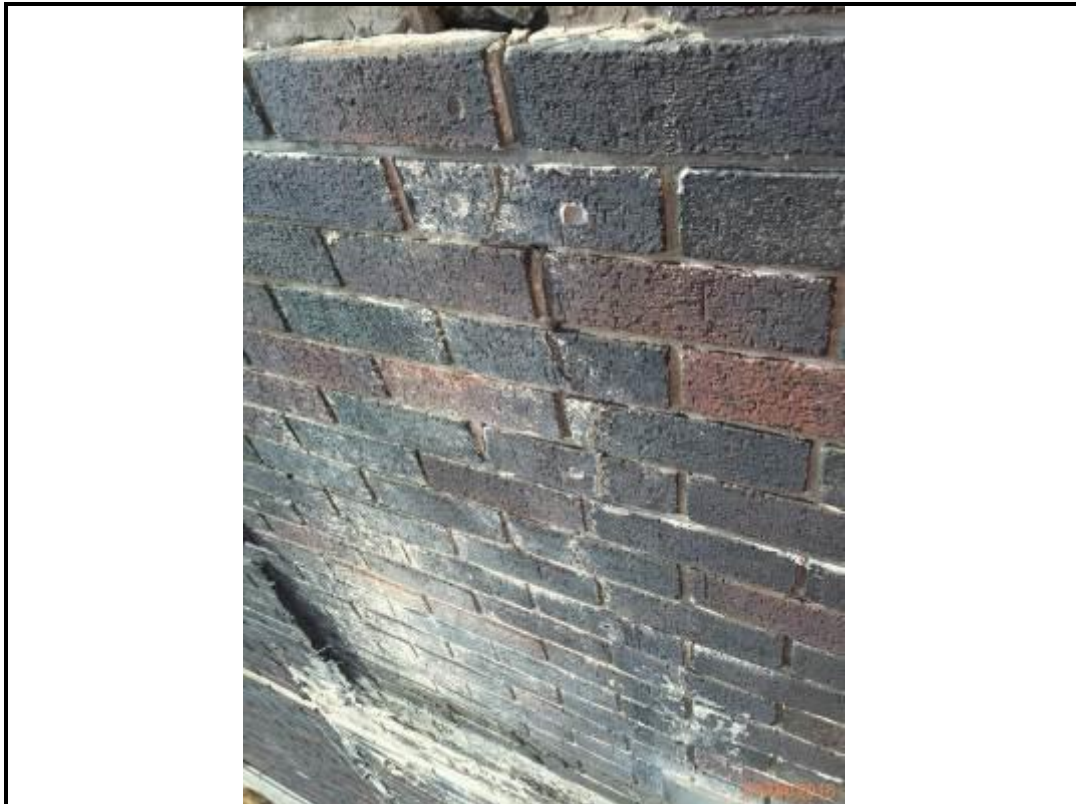


Photo 53. Drop 1 – South Elevation

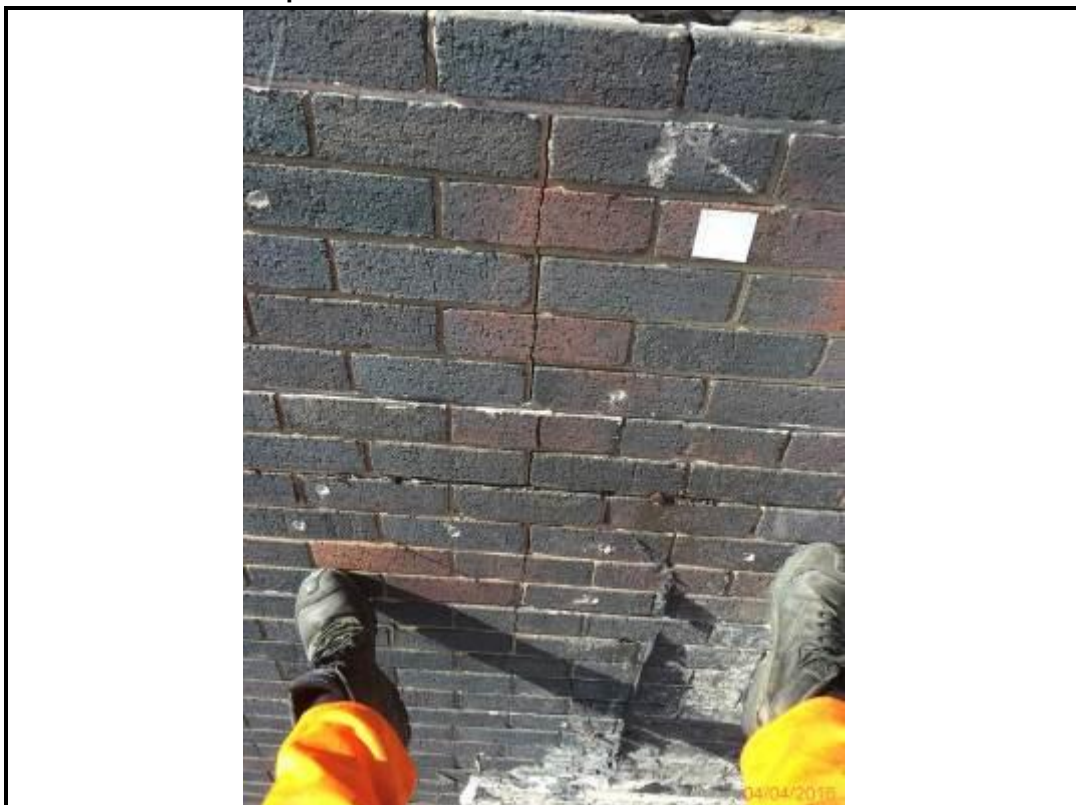


Photo 54. Drop 1 – South Elevation



Photo 55. Drop 1 – South Elevation

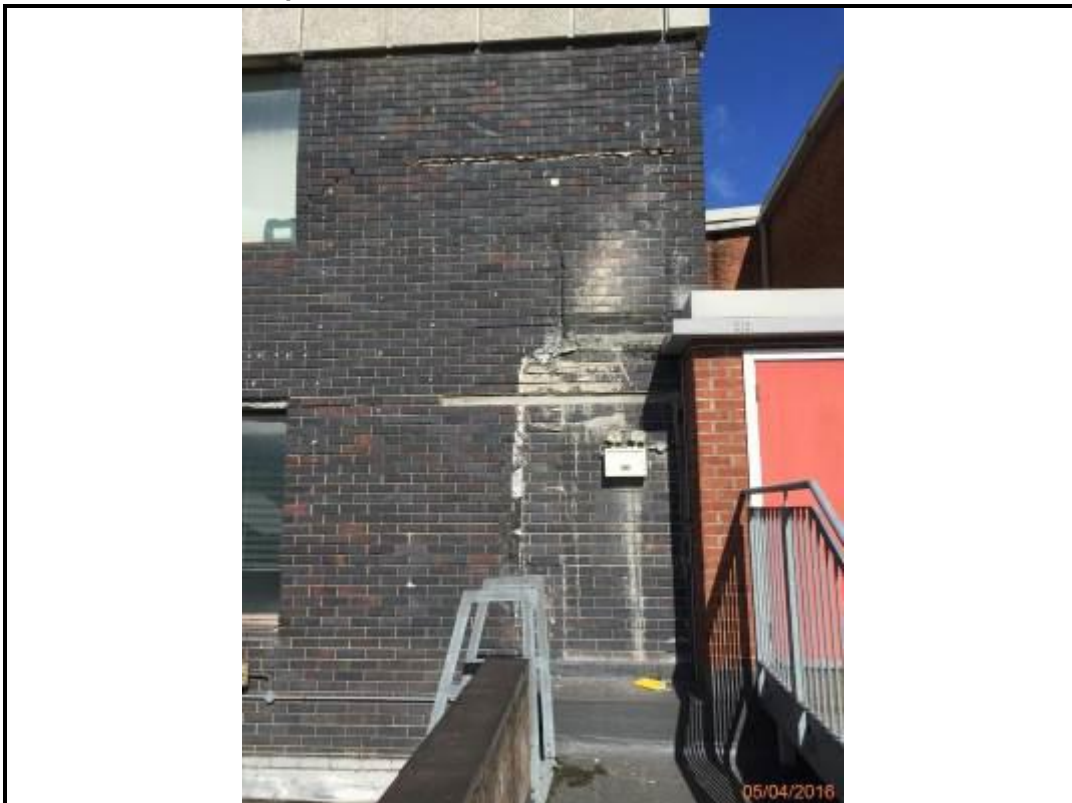


Photo 56. Drop 1 – South Elevation



Photo 57. Drop 1 – South Elevation



Photo 58. Drop 1 – South Elevation



Photo 59. Drop 2 – South Elevation

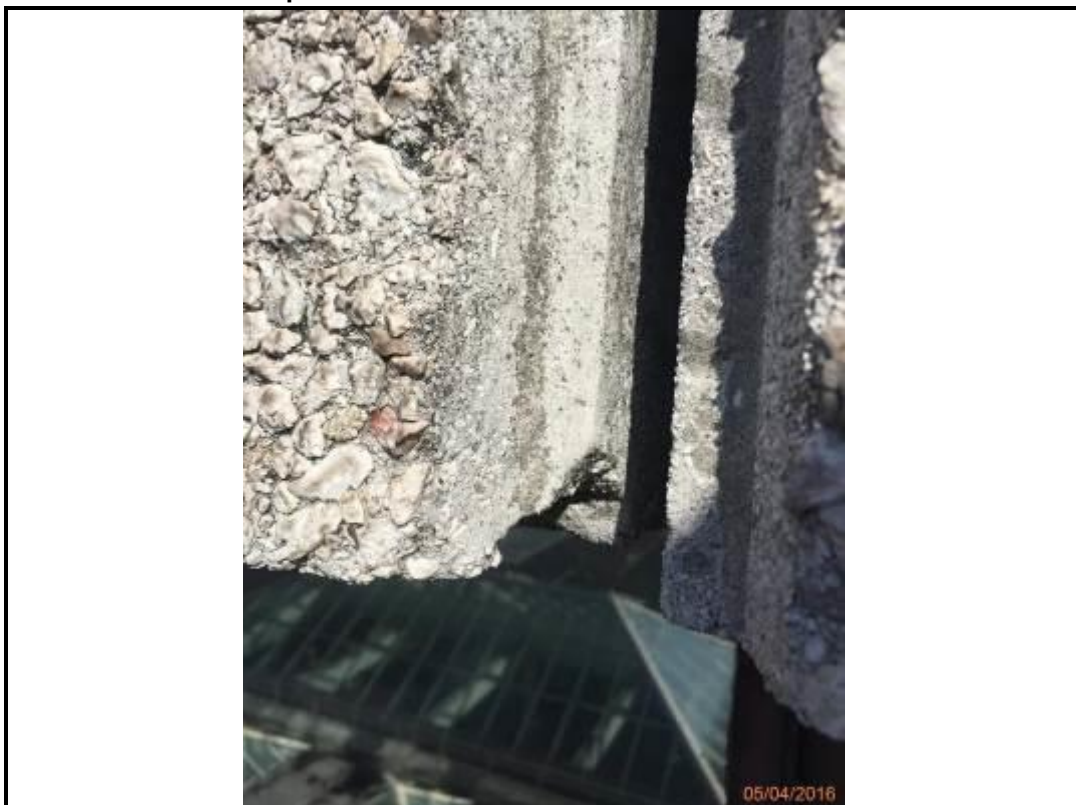


Photo 60. Drop 2 – South Elevation



Photo 61. Drop 2 – South Elevation

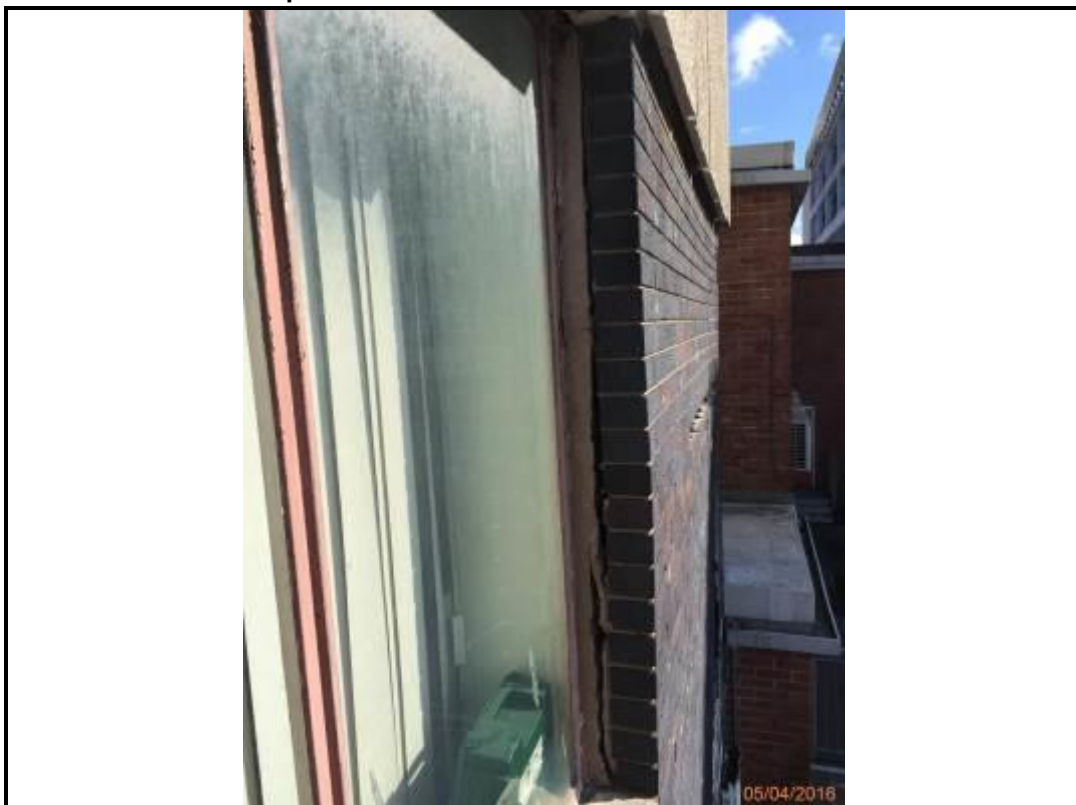


Photo 62. Drop 2 – South Elevation

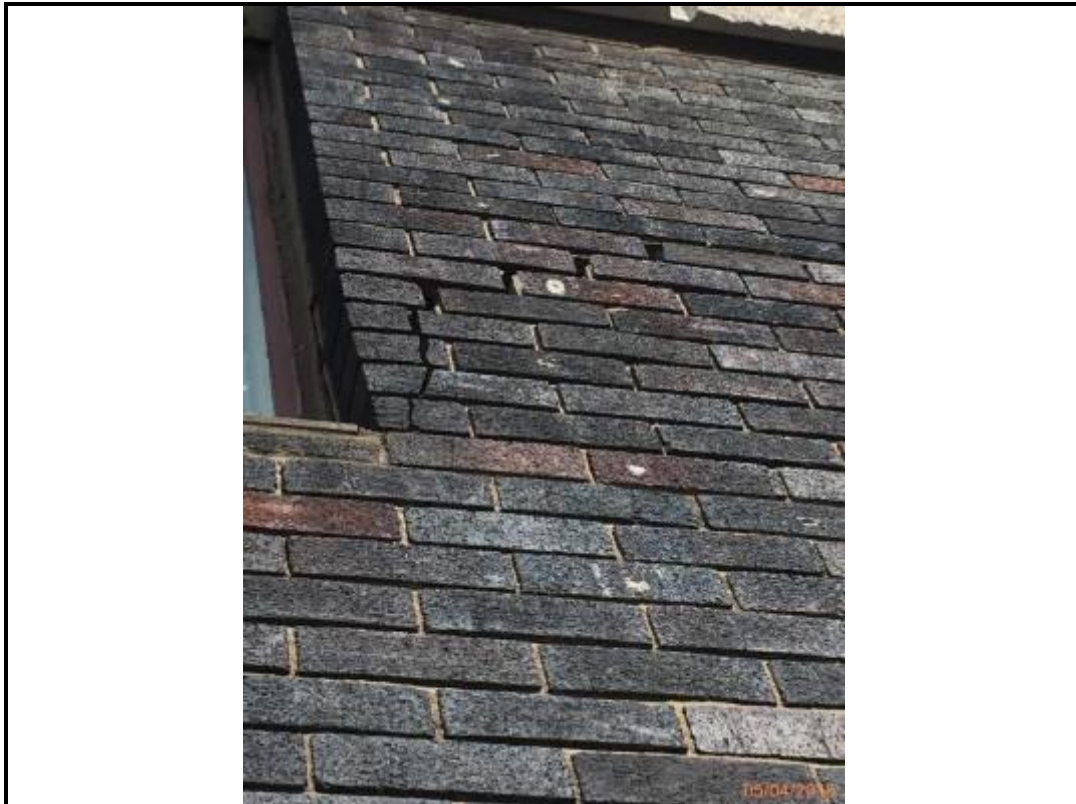


Photo 63. Drop 2 – South Elevation

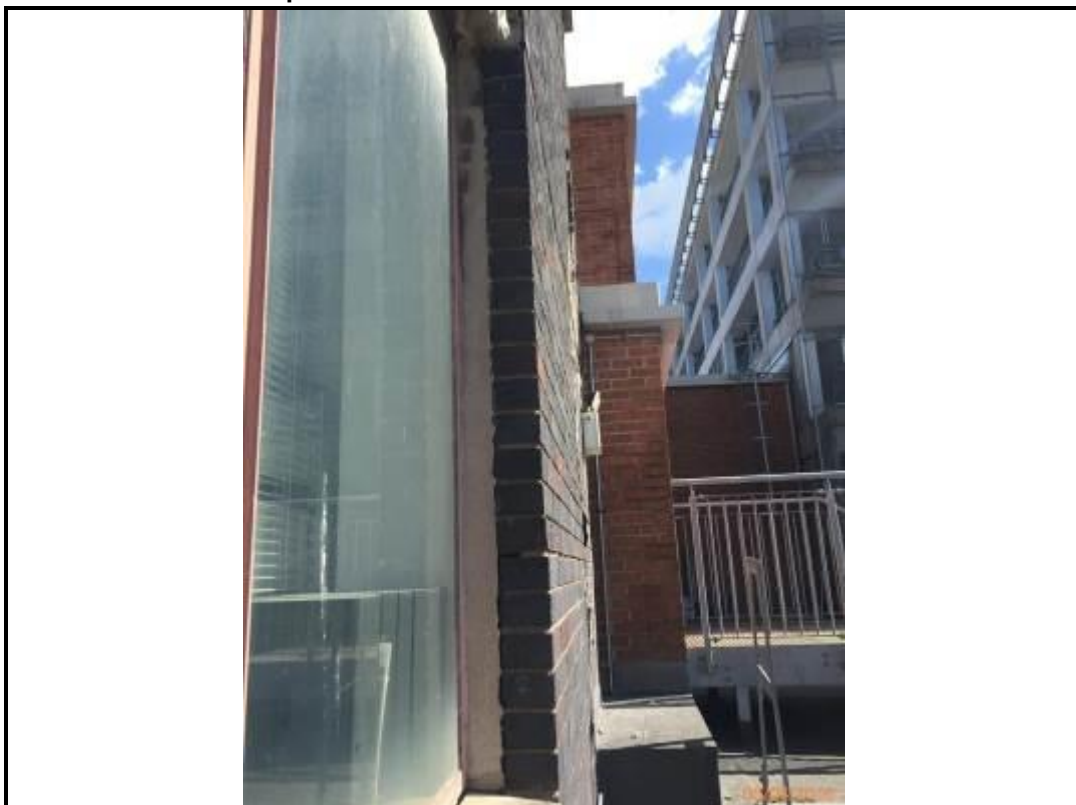


Photo 64. Drop 2 – South Elevation



Photo 65. Drop 2 – South Elevation



Photo 66. Drop 2 – South Elevation



Photo 67. Drop 2 – South Elevation

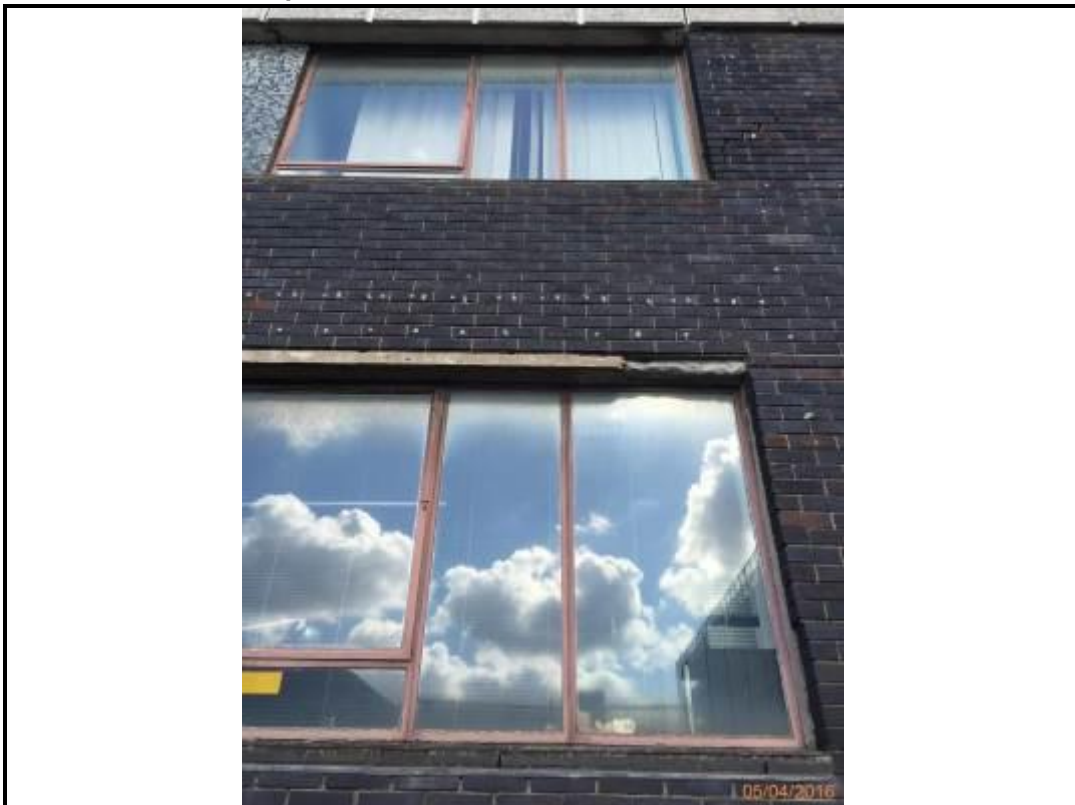


Photo 68. Drop 2 – South Elevation



Photo 69. Drop 3 – South Elevation



Photo 70. Drop 3 – South Elevation



Photo 71. Drop 3 – South Elevation



Photo 72. Drop 3 – South Elevation



Photo 73. Drop 3 – South Elevation



Photo 74. Drop 3 – South Elevation



Photo 75. Drop 3 – South Elevation



Photo 76. Drop 4 – South Elevation



Photo 77. Drop 5 – South Elevation



Photo 78. Drop 5 – South Elevation



Photo 79. Drop 5 – South Elevation

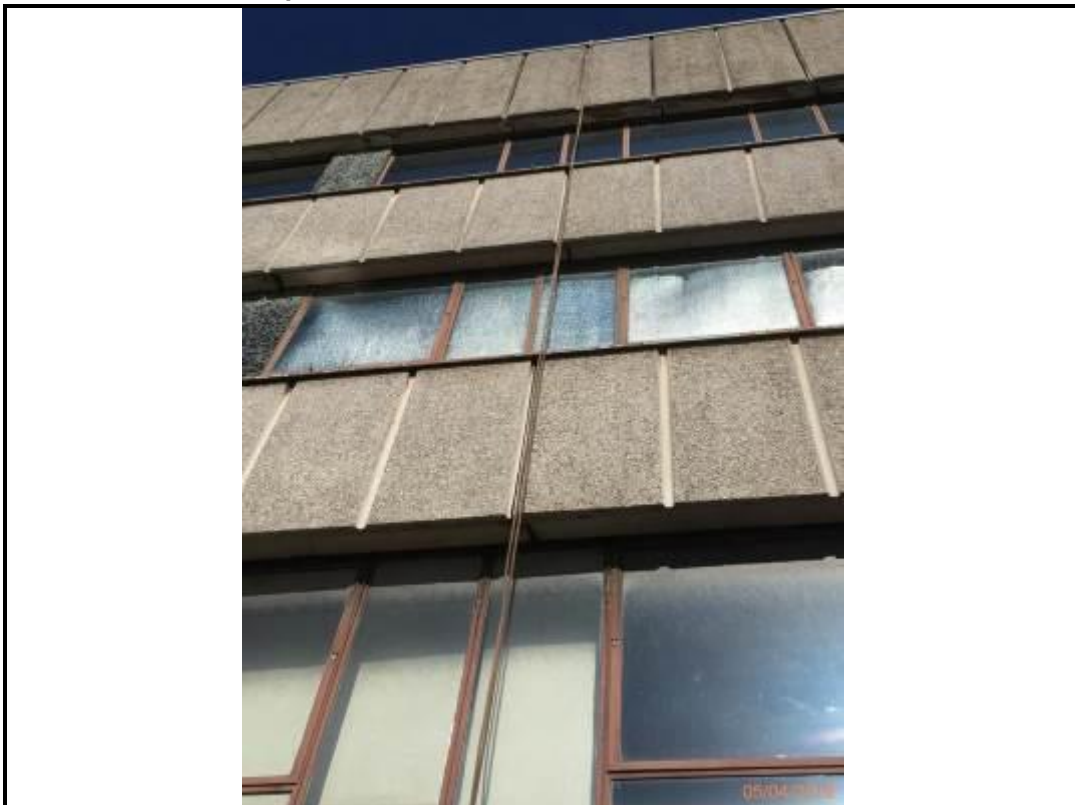


Photo 80. Drop 5 – South Elevation



Photo 81. Drop 6 –South Elevation



Photo 82. Drop 6 – South Elevation



Photo 83. Drop 6 – South Elevation



Photo 84. Drop 7 – South Elevation



Photo 85. Drop 7 – South Elevation



Photo 86. Drop 7 – South Elevation



Photo 87. Drop 7 – South Elevation



Photo 88. Drop 7 – South Elevation



Photo 89. Drop 8 – South Elevation



Photo 90. Drop 8 – South Elevation



Photo 91. Drop 8 – South Elevation



Photo 92. Drop 8 – South Elevation



Photo 93. Drop 8 – South Elevation



Photo 94. Drop 8 – South Elevation



Photo 95. Drop 9 – South Elevation



Photo 96. Drop 9 – South Elevation



Photo 97. Drop 9 – South Elevation



Photo 98. Drop 9 – South Elevation



Photo 99. Drop 10 – South Elevation



Photo 100. Drop 10 – South Elevation



Photo 101. Drop 10 – South Elevation



Photo 102. Drop 10 – South Elevation



Photo 103. Drop 11 – South Elevation



Photo 104. Drop 11 – South Elevation



Photo 105. Drop 12 – South Elevation



Photo 106. Drop 12 – South Elevation



Photo 107. Drop 12 – South Elevation



Photo 108. Drop 12 – South Elevation



Photo 109. Drop 13 – South Elevation



Photo 110. Drop 13 – South Elevation



Photo 111. Drop 13 – South Elevation



Photo 112. Drop 13 – South Elevation



Photo 113. Drop 13 – South Elevation



Photo 114. Drop 13 – South Elevation



Photo 115. Drop 13 – South Elevation



Photo 116. Drop 13 – South Elevation



Photo 117. Drop 13 – South Elevation



Photo 118. Drop 13 – South Elevation



Photo 119. Drop 13 – South Elevation



Photo 120. Drop 13 – South Elevation

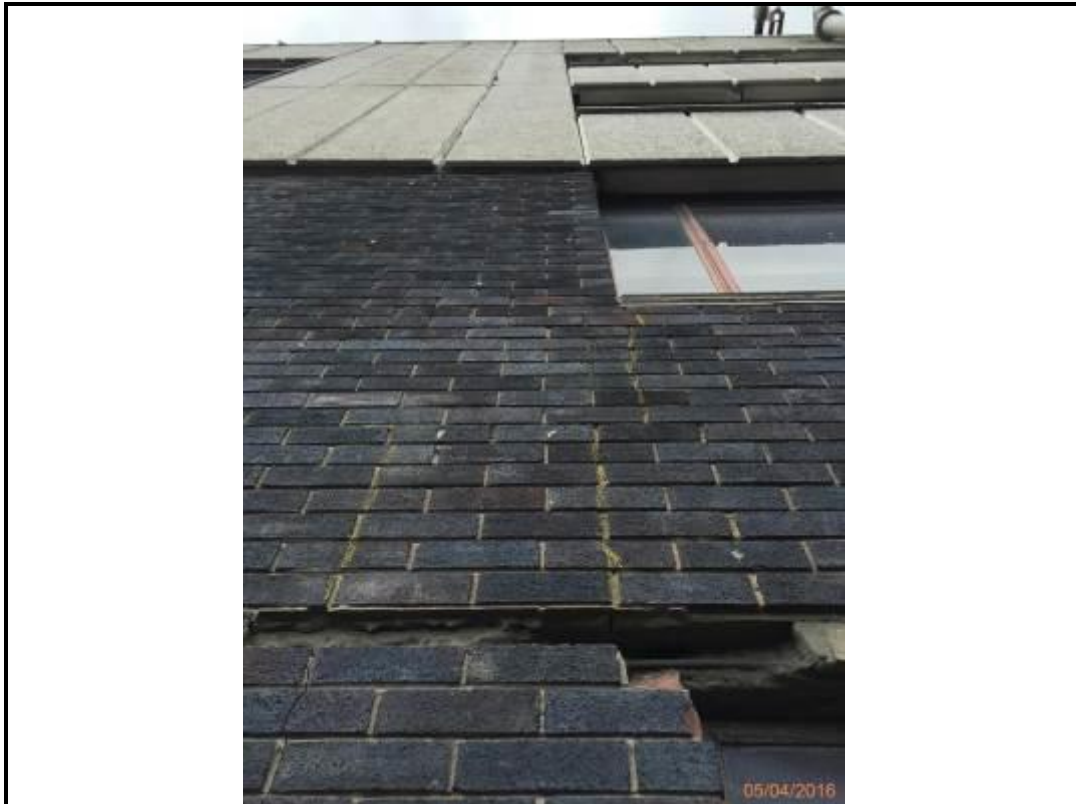


Photo 121. Drop 13 – South Elevation



Photo 122. Drop 14 – South Elevation



Photo 123. Drop 14 – South Elevation



Photo 124. Drop 14 – South Elevation



Photo 125. Drop 14 – South Elevation



Photo 126. Drop 14 – South Elevation



Photo 127. Drop 14 – South Elevation



Photo 128. Drop 14 – South Elevation



Photo 129. Drop 14 – South Elevation



Photo 130. Drop 14 – South Elevation



Photo 131. Drop 14 – South Elevation



Photo 132. Drop 14 – South Elevation



Photo 133. Drop 14 – South Elevation



Photo 134. Drop 14 – South Elevation



Photo 135. Drop 15 – South Elevation



Photo 136. Drop 15 – South Elevation



Photo 137. Drop 15 – South Elevation



Photo 138. Drop 15 – South Elevation

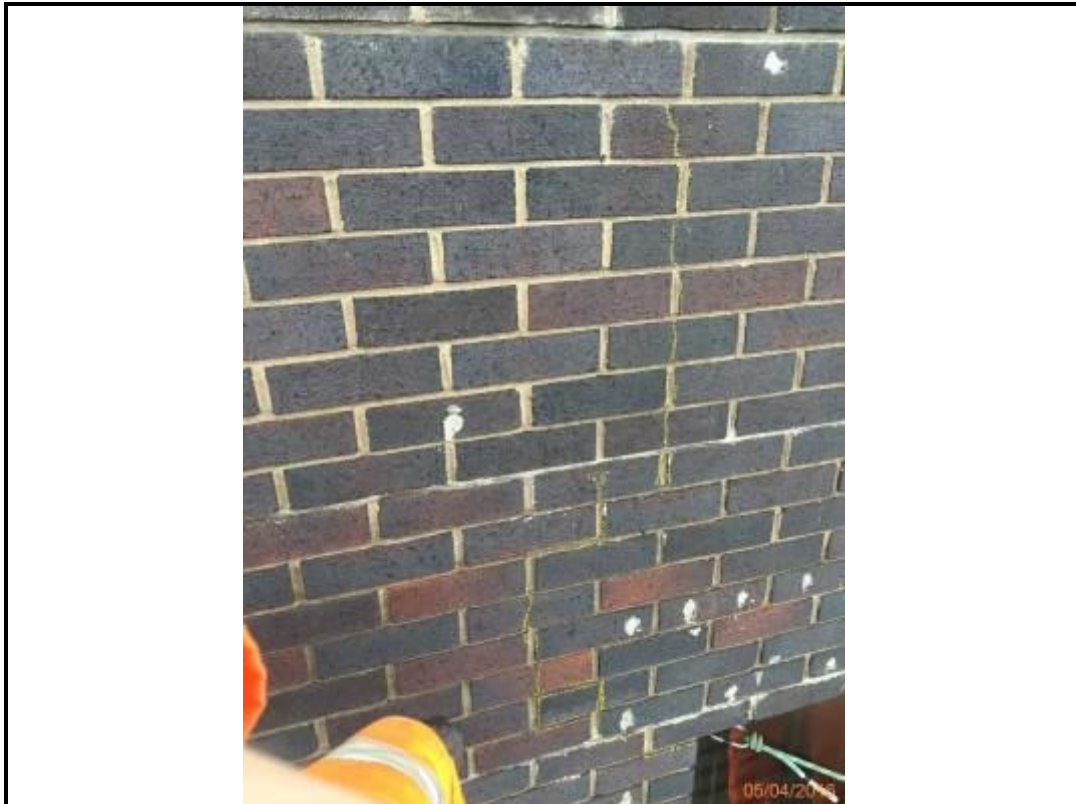


Photo 139. Drop 15 – South Elevation

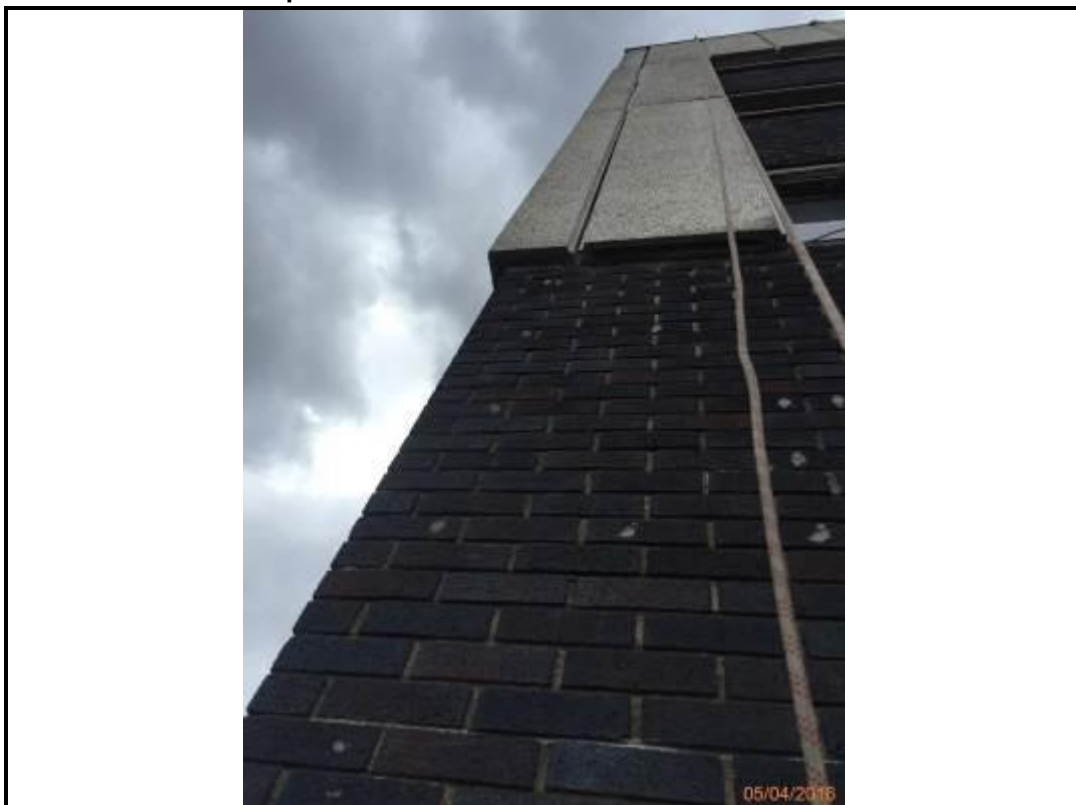


Photo 140. Drop 15 – South Elevation



Photo 141. East Elevation – Brickwork Cladding Beneath Overhang



Photo 142. East Elevation – Brickwork Cladding Beneath Overhang



Photo 143. East Elevation – Brickwork Cladding Beneath Overhang



Photo 144. East Elevation – Brickwork Cladding Beneath Overhang



Photo 145. East Elevation – Brickwork Cladding Beneath Overhang



Photo 146. East Elevation – Brickwork Cladding Beneath Overhang



Photo 147. East Elevation – Brickwork Cladding Beneath Overhang



Photo 148. East Elevation – Brickwork Cladding Beneath Overhang



Photo 149. East Elevation – Brickwork Cladding Beneath Overhang

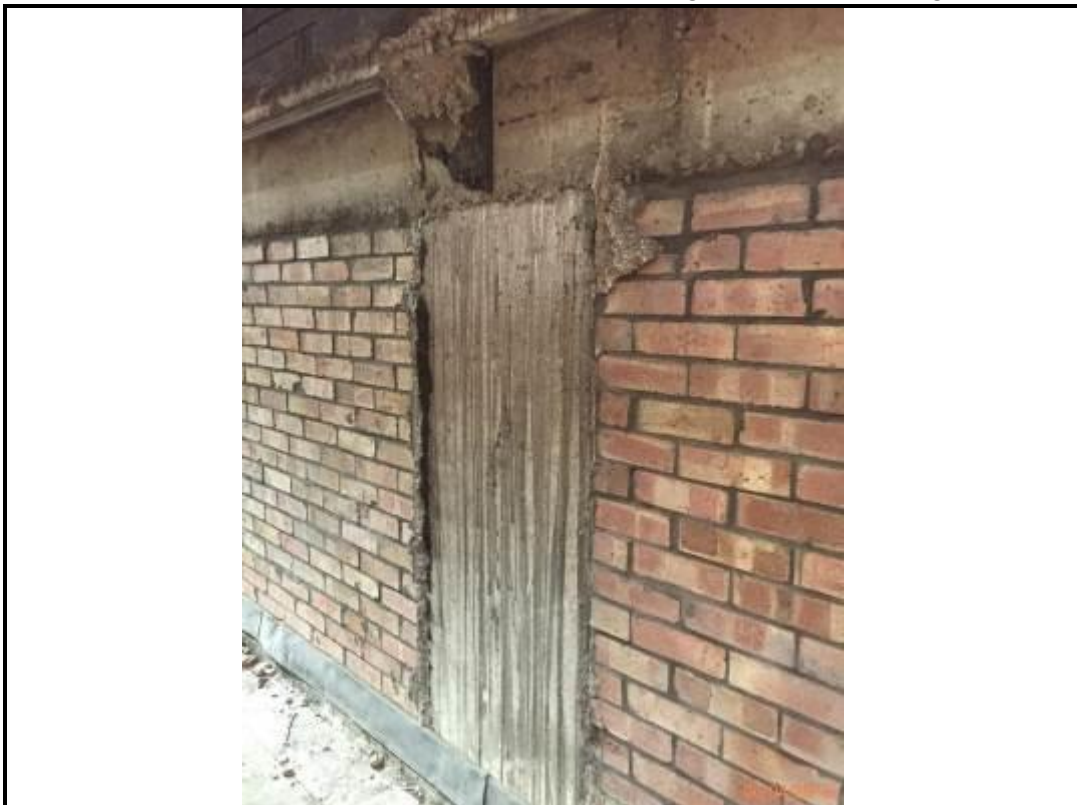


Photo 150. East Elevation – Brickwork Cladding Beneath Overhang



Photo 151. East Elevation – Brickwork Cladding Beneath Overhang

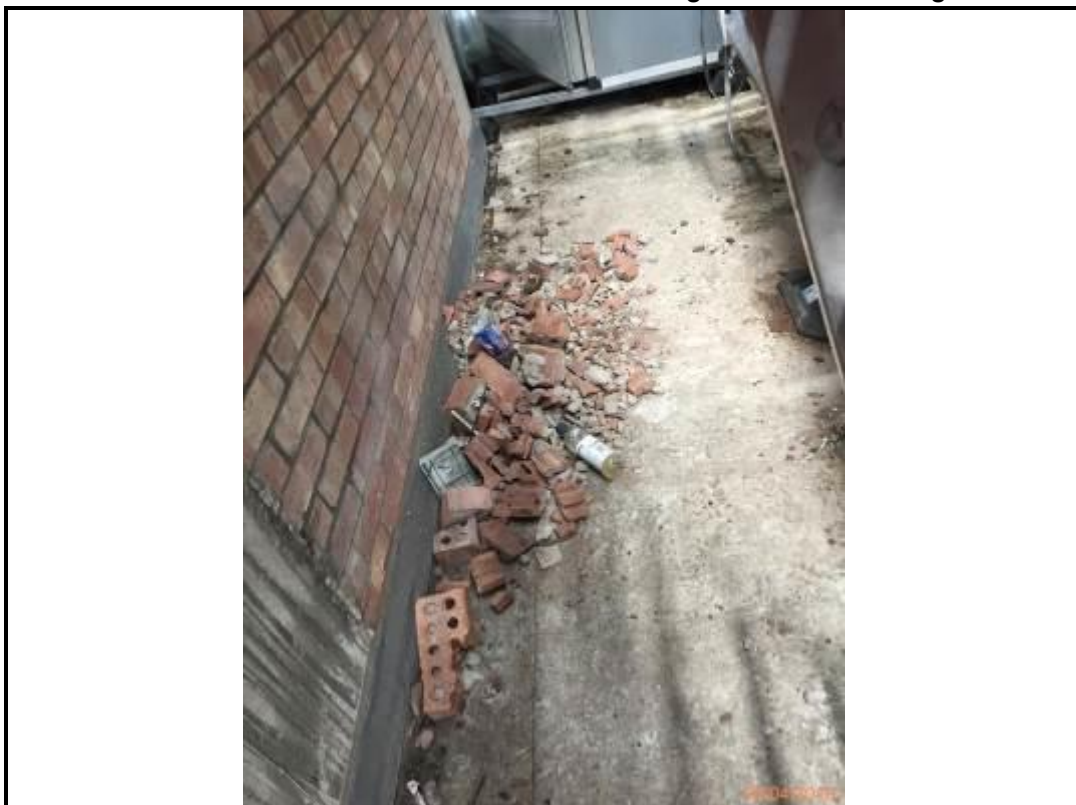


Photo 152. East Elevation – Brickwork Cladding Beneath Overhang



Photo 153. East Elevation – Brickwork Cladding Beneath Overhang



Photo 154. East Elevation – Brickwork Cladding Beneath Overhang



Photo 155. East Elevation – Brickwork Cladding Beneath Overhang



Photo 156. East Elevation – Brickwork Cladding Beneath Overhang



Photo 157. East Elevation – Brickwork Cladding Beneath Overhang



Photo 158. East Elevation – Brickwork Cladding Beneath Overhang



Photo 159. East Elevation – Brickwork Cladding Beneath Overhang



Photo 160. East Elevation – Brickwork Cladding Beneath Overhang



Photo 161. East Elevation – Brickwork Cladding Beneath Overhang



Photo 162. East Elevation – Brickwork Cladding Beneath Overhang



Photo 163. East Elevation – Brickwork Cladding Beneath Overhang



Photo 164. East Elevation – Brickwork Cladding Beneath Overhang



Photo 165. East Elevation – Brickwork Cladding Beneath Overhang



Photo 166. Drop 1 – West Elevation



Photo 167. Drop 1 – West Elevation



Photo 168. Drop 1 – West Elevation



Photo 169. Drop 1 – West Elevation



Photo 170. Drop 1 – West Elevation



Photo 171. Drop 1 – West Elevation



Photo 172. Drop 1 – West Elevation



Photo 173. Drop 1 – West Elevation



Photo 174. Drop 2 – West Elevation



Photo 175. Drop 2 – West Elevation

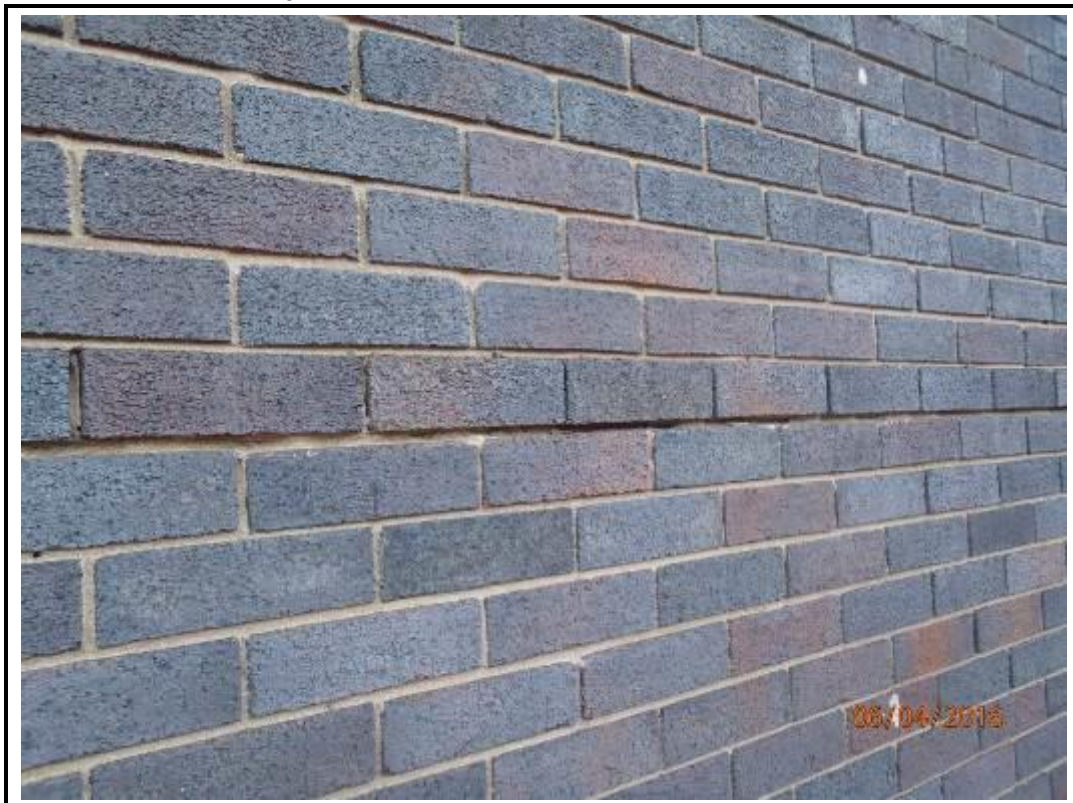


Photo 176. Drop 2 – West Elevation



Photo 177. Drop 2 – West Elevation



Photo 178. Drop 3 – West Elevation



Photo 179. Drop 3 – West Elevation



Photo 180. Drop 3 – West Elevation



Photo 181. Drop 3 – West Elevation



Photo 182. Drop 3 – West Elevation



Photo 183. Drop 4 – West Elevation

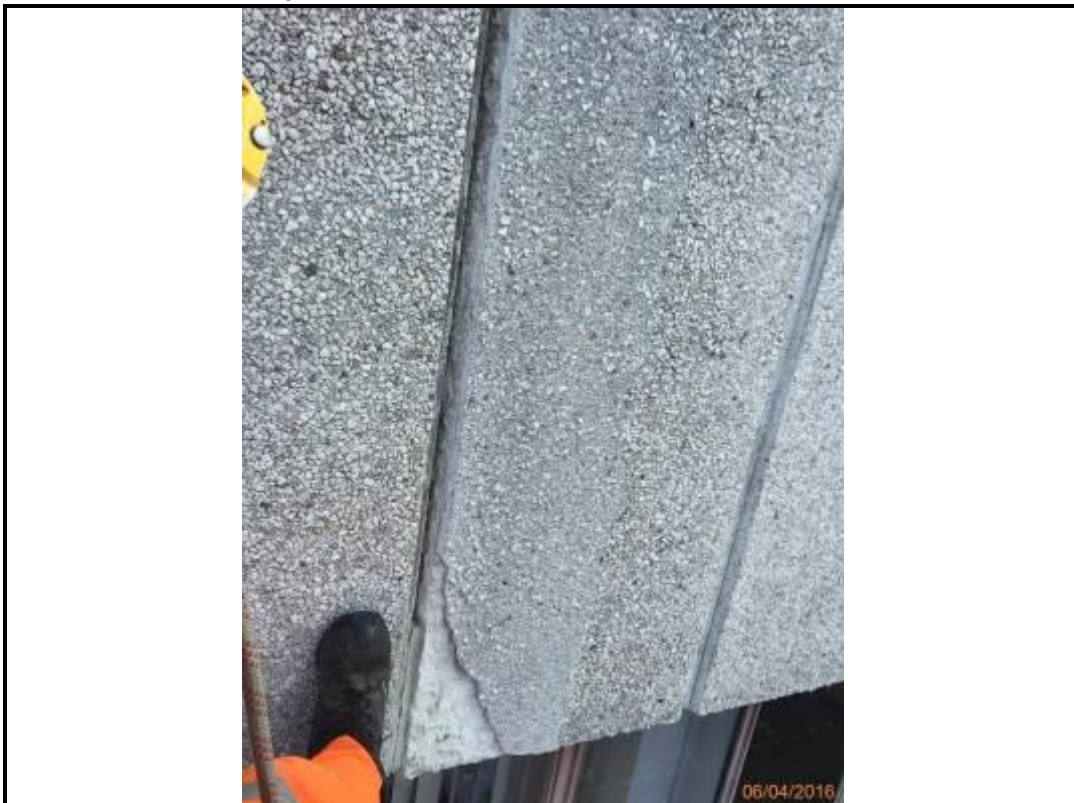


Photo 184. Drop 4 – West Elevation

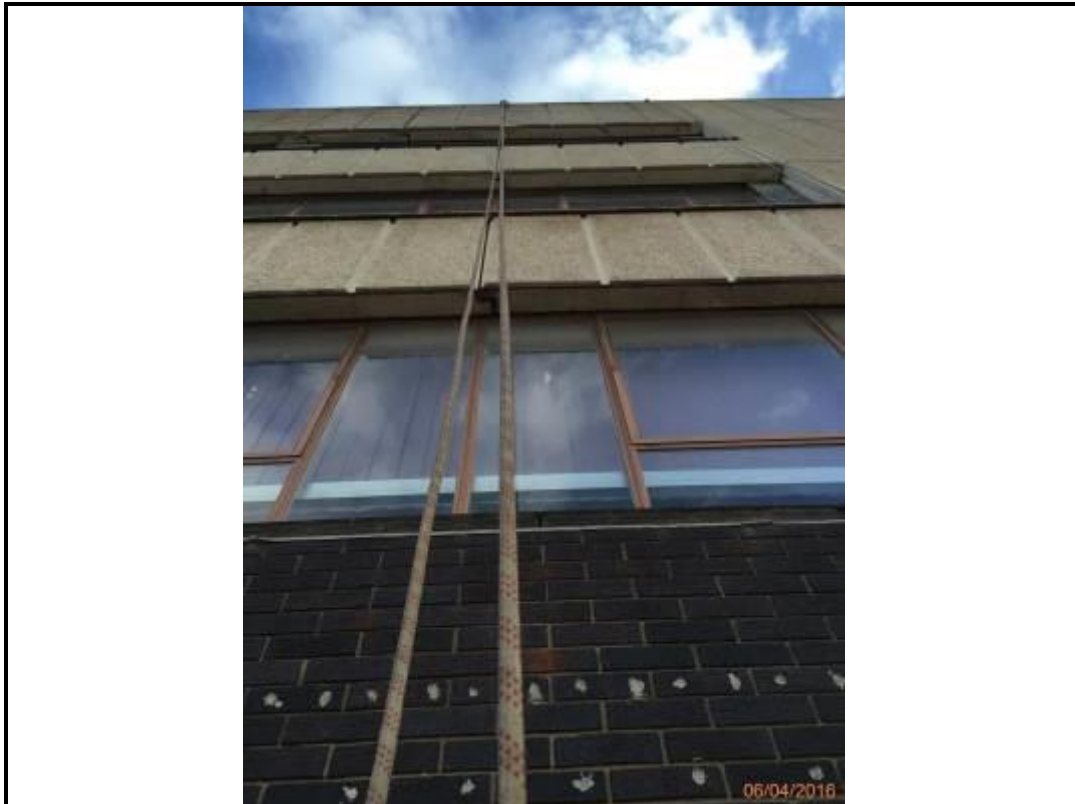


Photo 185. Drop 4 – West Elevation

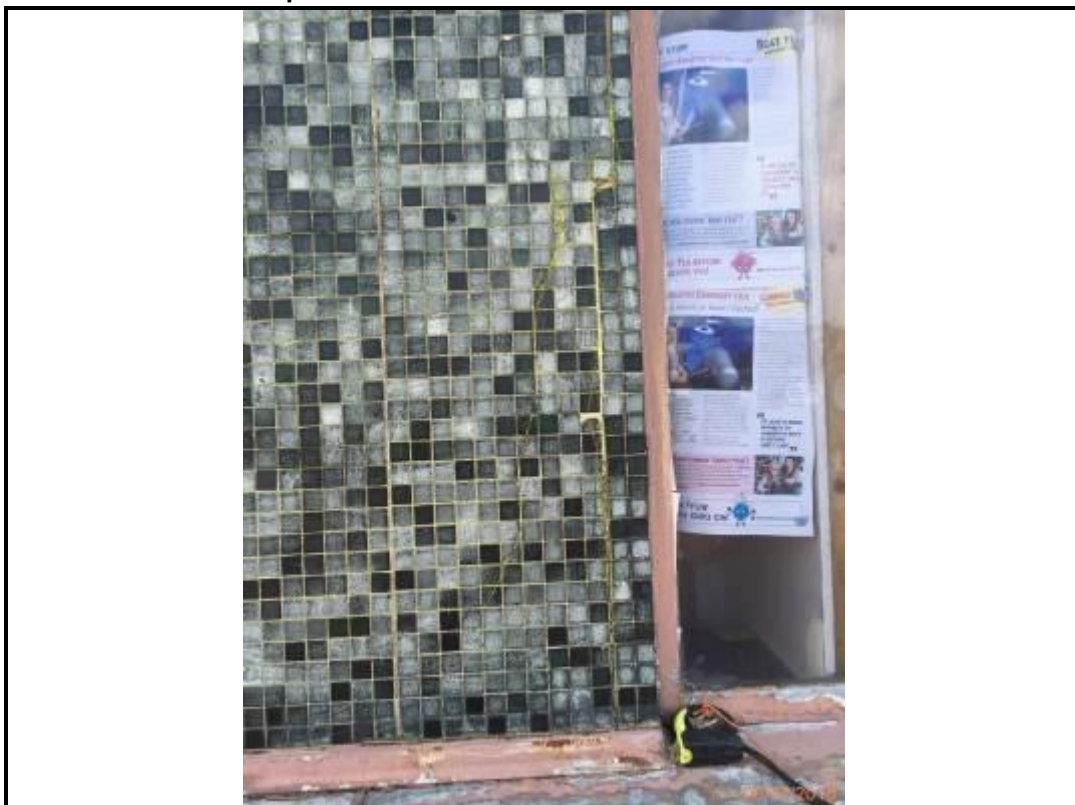


Photo 186. Drop 5 – West Elevation



Photo 187. Drop 5 – West Elevation



Photo 188. Drop 5 – West Elevation



Photo 189. Drop 5 – West Elevation

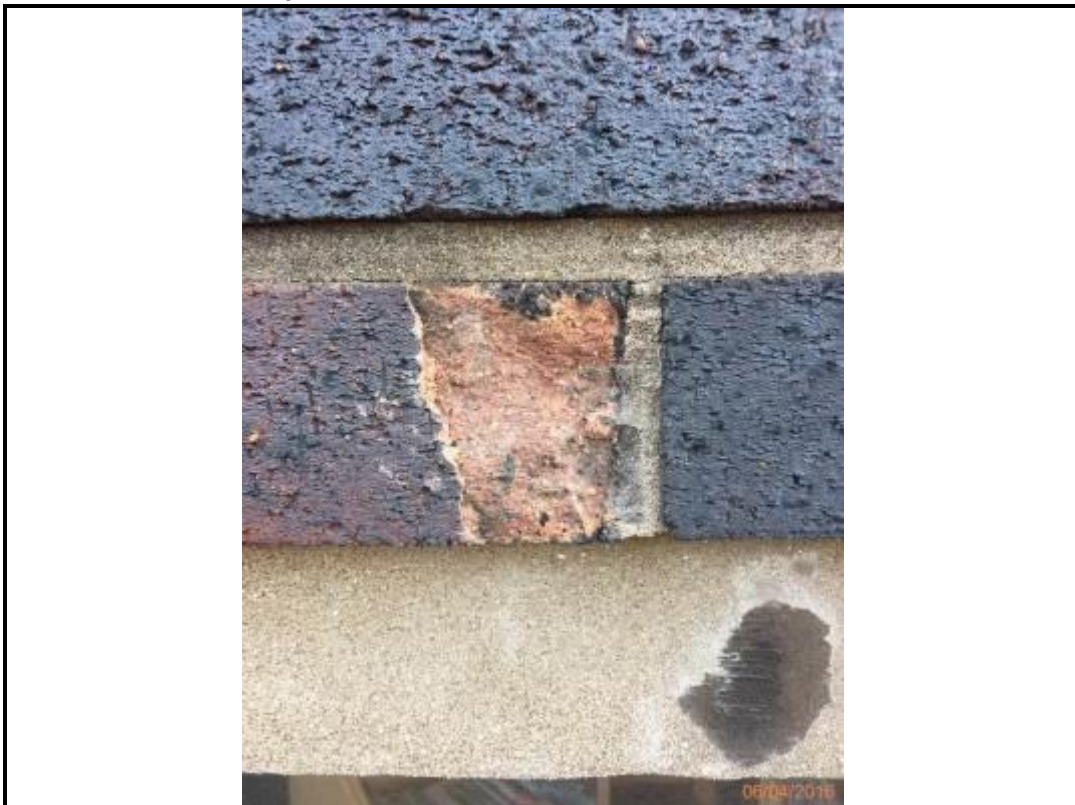


Photo 190. Drop 5 – West Elevation

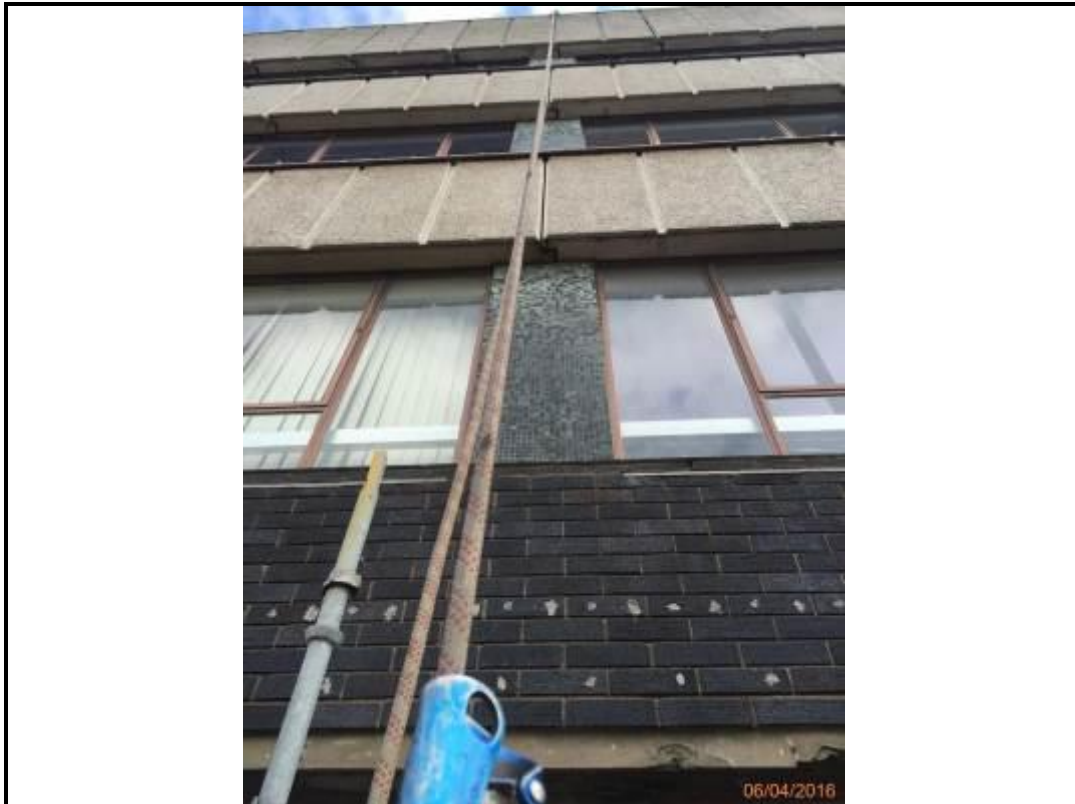


Photo 191. Drop 5 – West Elevation



Photo 192. Drop 6 – West Elevation



Photo 193. Drop 6 – West Elevation



Photo 194. Drop 7 – West Elevation



Photo 195. Drop 7 – West Elevation



Photo 196. Drop 7 – West Elevation



Photo 197. Drop 7 – West Elevation

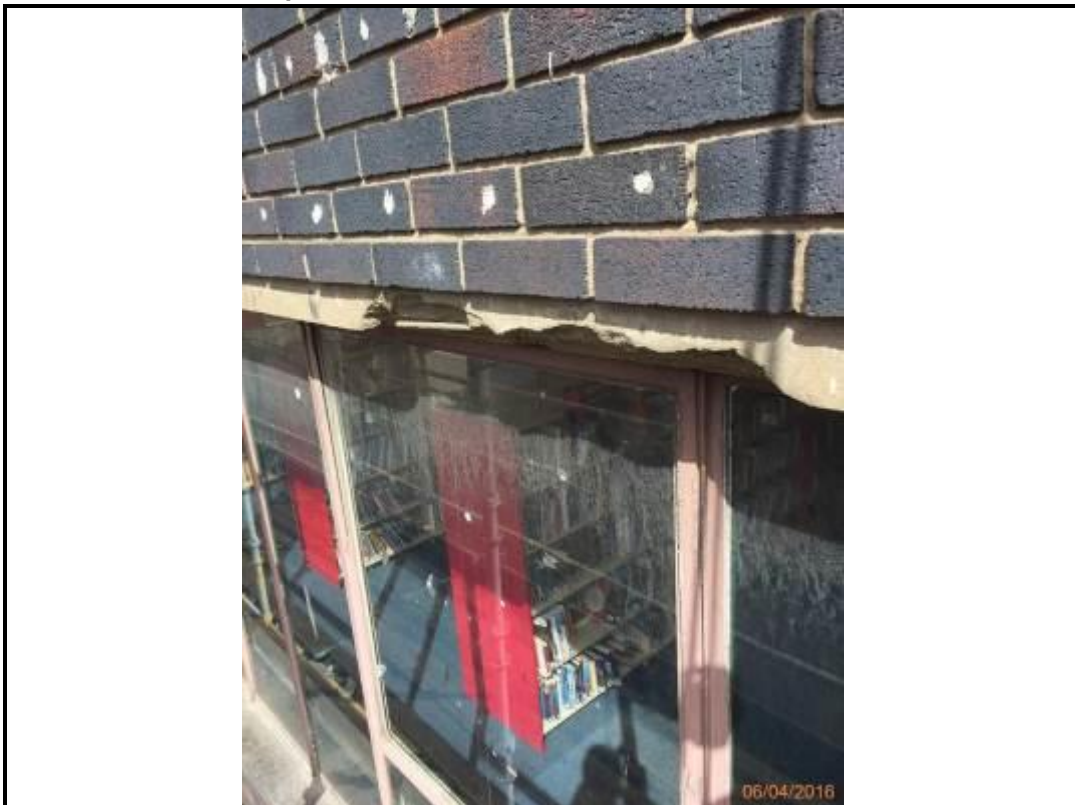


Photo 198. Drop 7 – West Elevation



Photo 199. Drop 7 – West Elevation



Photo 200. Drop 8 – West Elevation

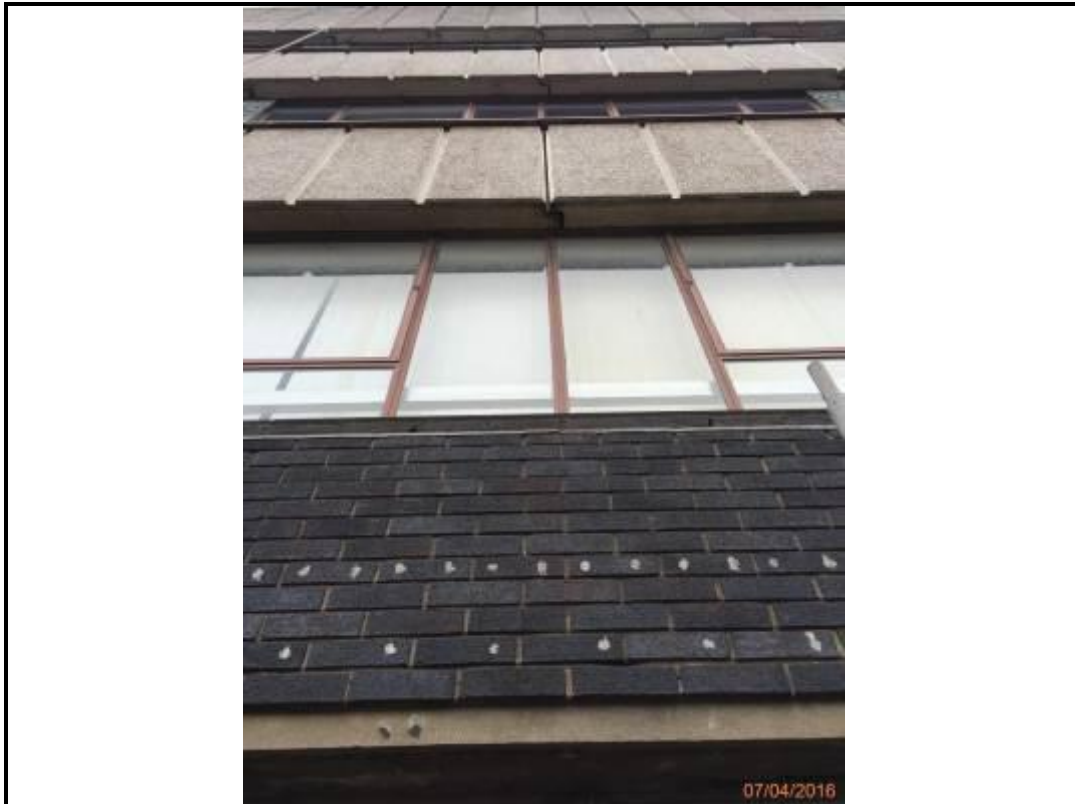


Photo 201. Drop 8 – West Elevation



Photo 202. Drop 9 – West Elevation



Photo 203. Drop 9 – West Elevation



Photo 204. Drop 9 – West Elevation



Photo 205. Drop 10 – West Elevation



Photo 206. Drop 10 – West Elevation



Photo 207. Drop 11 – West Elevation



Photo 208. Drop 11 – West Elevation



Photo 209. Drop 11 – West Elevation



Photo 210. Drop 11 – West Elevation



Photo 211. Drop 11 – West Elevation



Photo 212. Drop 11 – West Elevation



Photo 213. Drop 11 – West Elevation



Photo 214. Drop 11 – West Elevation



Photo 215. Drop 11 – West Elevation



Photo 216. Drop 11 – West Elevation



Photo 217. Drop 12 – West Elevation



Photo 218. Drop 12 – West Elevation



Photo 219. Drop 12 – West Elevation



Photo 220. Drop 12 – West Elevation



Photo 221. Drop 12 – West Elevation



Photo 222. Drop 13 – West Elevation



Photo 223. Drop 13 – West Elevation



Photo 224. Drop 13 – West Elevation



Photo 225. Drop 13 – West Elevation



Photo 226. Drop 13 – West Elevation



Photo 227. Drop 13 – West Elevation



Photo 228. Drop 13 – West Elevation



Photo 229. Drop 1 – North Elevation



Photo 230. Drop 1 – North Elevation



Photo 231. Drop 1 – North Elevation



Photo 232. Drop 2 – North Elevation



Photo 233. Drop 2 – North Elevation



Photo 234. Drop 3 – North Elevation



Photo 235. Drop 3 – North Elevation



Photo 236. Drop 3 – North Elevation



Photo 237. Drop 4 – North Elevation



Photo 238. Drop 4 – North Elevation



Photo 239. Drop 4 – North Elevation



Photo 240. Drop 4 – North Elevation



Photo 241. Drop 4 – North Elevation



Photo 242. Drop 5 – North Elevation



Photo 243. Drop 5 – North Elevation



Photo 244. Drop 5 – North Elevation



Photo 245. Drop 6 – North Elevation



Photo 246. Drop 6 – North Elevation



Photo 247. Drop 6 – North Elevation



Photo 248. Drop 7 – North Elevation



Photo 249. Drop 7 – North Elevation



Photo 250. Drop 7 – North Elevation



Photo 251. Drop 8 – North Elevation



Photo 252. Drop 8 – North Elevation



Photo 253. Drop 8 – North Elevation



Photo 254. Drop 9 – North Elevation



Photo 255. Drop 9 – North Elevation

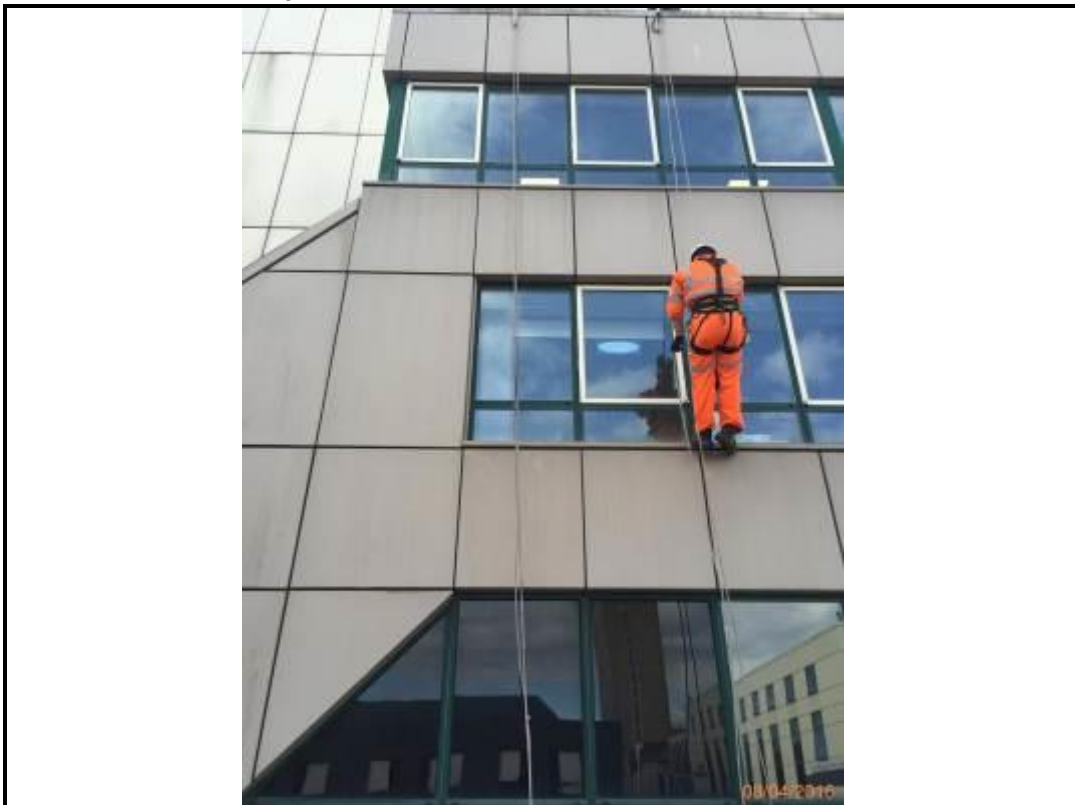


Photo 256. Drop 9 – North Elevation



Photo 257. Drop 10 – North Elevation



Photo 258. Drop 10 – North Elevation



Photo 259. Drop 11 – North Elevation



Photo 260. Drop 11 – North Elevation



Photo 261. Drop 12 – North Elevation



Photo 262. Drop 12 – North Elevation

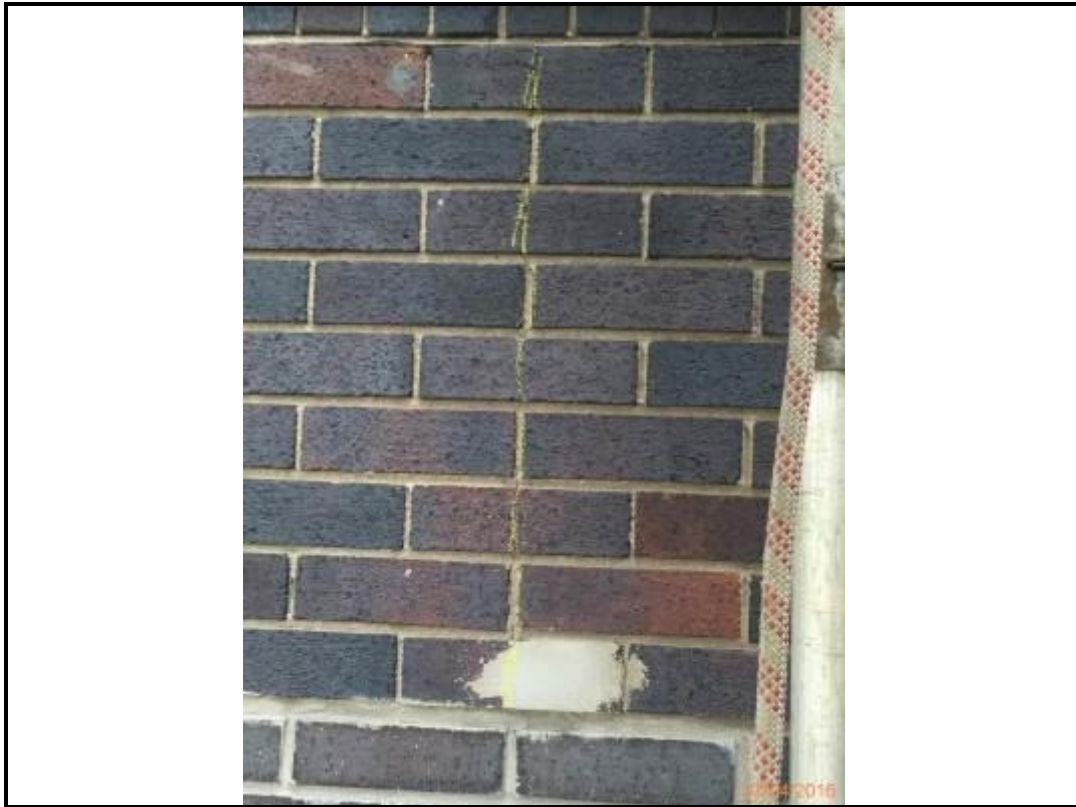


Photo 263. Drop 12 – North Elevation

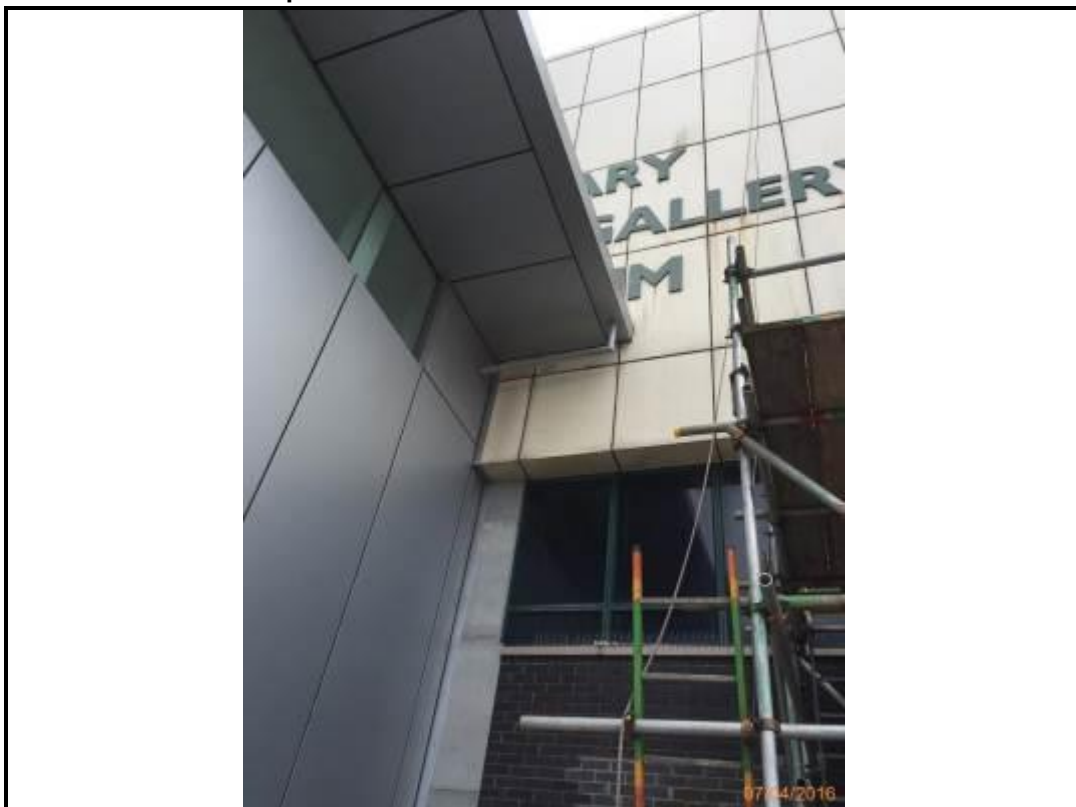


Photo 264. Drop 13 – North Elevation



Photo 265. Drop 13 – North Elevation



Photo 266. Drop 13 – North Elevation



Photo 267. Drop 13 – North Elevation



Photo 268. Drop 14 – North Elevation

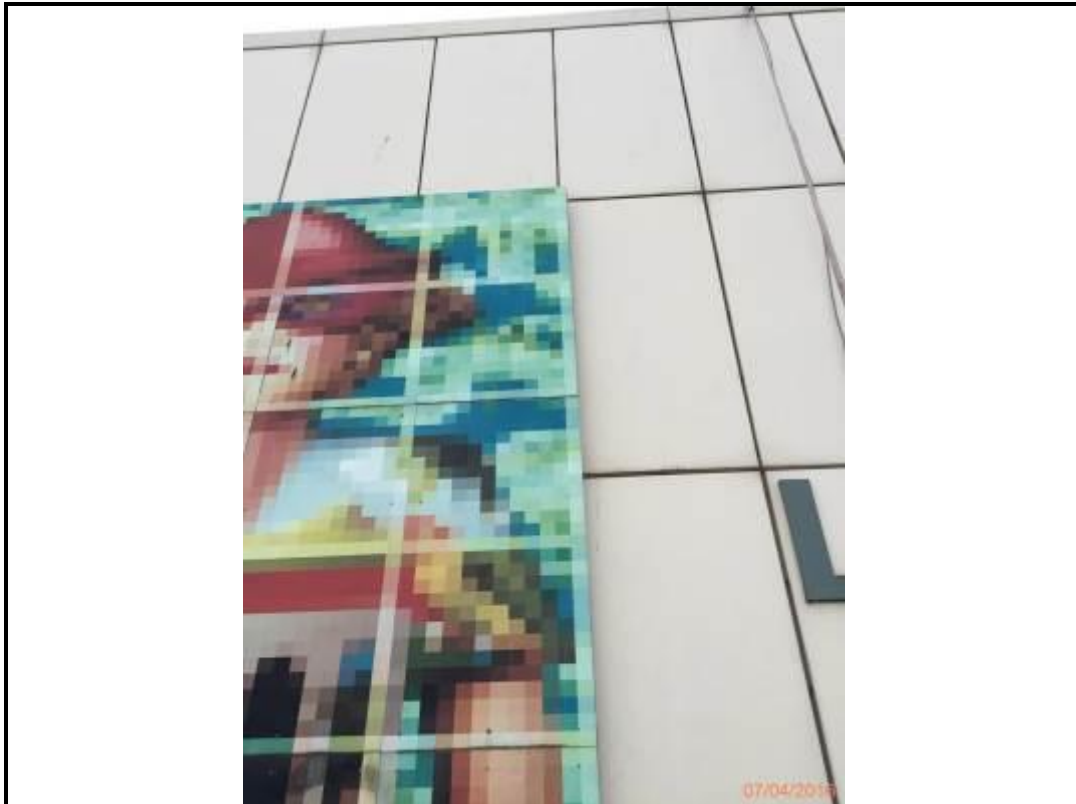


Photo 269. Drop 15 – North Elevation

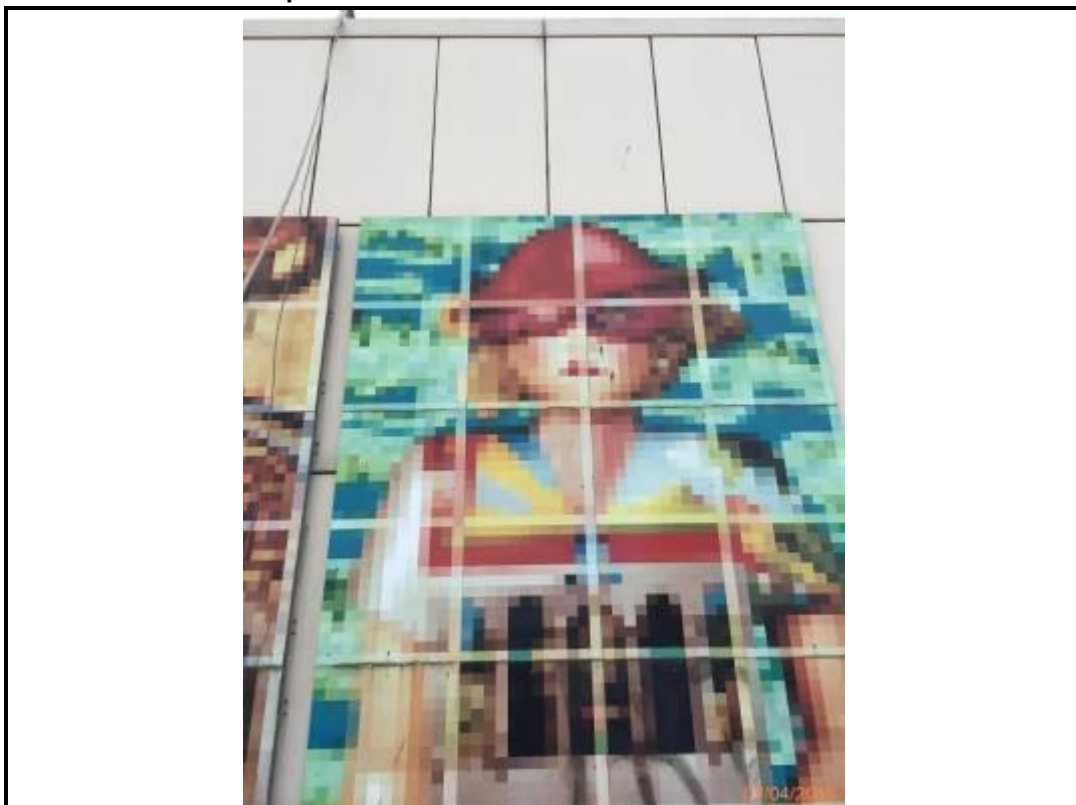


Photo 270. Drop 16 – North Elevation



Photo 271. Drop 17 – North Elevation



Photo 272. Drop 18 – North Elevation



Photo 273. Drop 19 – North Elevation

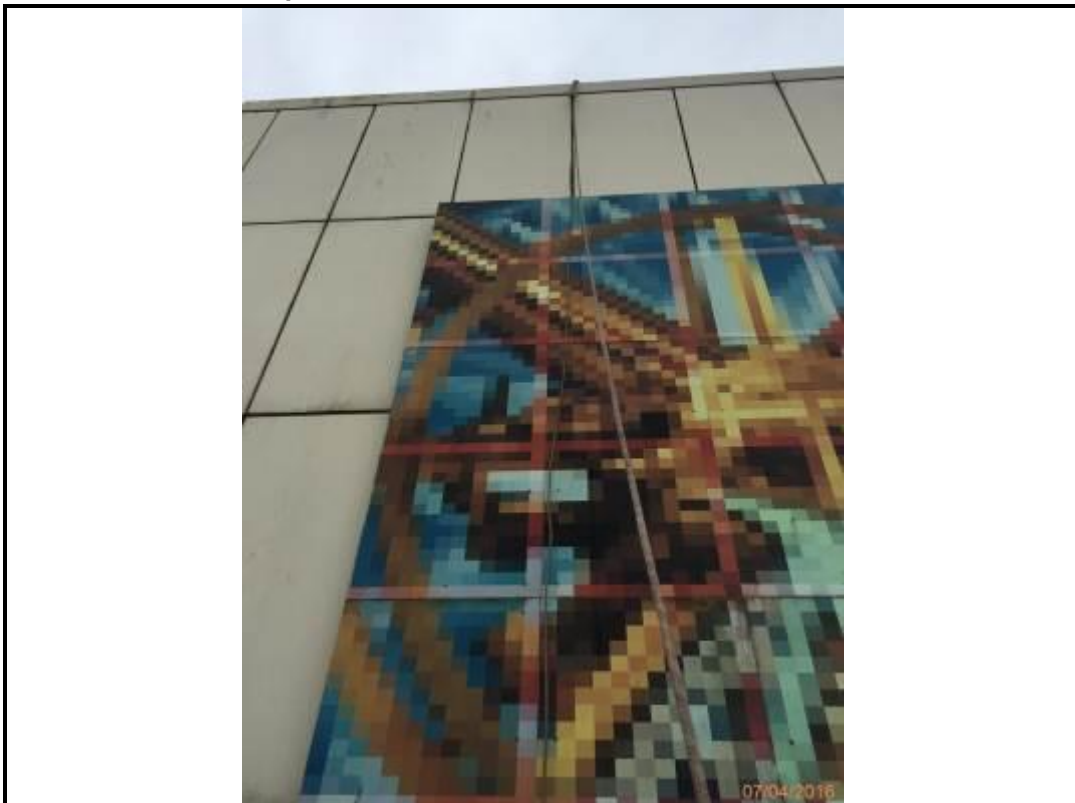


Photo 274. Drop 20 – North Elevation



Photo 275. Drop 21 – North Elevation

APPENDIX

iii) CONCRETE TESTING REPORT

STRUCTURAL CONCRETE INVESTIGATIONS

AT

NEWPORT CENTRAL LIBRARY

NEWPORT

WALES



Prepared For:

Shear Design Limited
7 Ashtree Court
Woodsy Close
Cardiff Business Park
Cardiff CF23 8RW

Date: 17th June 2016

Report Ref. 16040/1

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2.0	DESCRIPTION OF PROPERTY.	1
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Appendices

Appendix A – Photographs

Appendix B – Conditions and Limitations

1. Introduction

RS Specialist Services were instructed to undertake an investigation of the structural concrete at Newport Central Library, Newport, Wales.

The purpose of the investigation was to:

- Assess the form of concrete construction.
- Remove samples of concrete from accessible elements for chloride and High Alumina Cement (HAC) laboratory analysis.
- Conduct additional laboratory analysis on samples removed by others.
- Carry out testing of concrete for durability purposes (concrete cover and carbonation).

A visit was made to the property on 13th June 2016.

The observations and comments contained in this report relate to the findings at that time.

The investigation was carried out by Richard Sharp BSc CEng MINDT MIAQ.

2. Property Description

The library is a large 1960s building arranged over ground and 4 upper floor levels.

The structure of the property comprises insitu cast concrete columns and beams supporting insitu cast concrete floors.

The external elevations comprise a combination of the following:

- Brickwork
- Precast cladding panels
- Glazing
- Mosaic tiling

3. Method of Investigation

3.1 Structure

A walk around visual inspection was undertaken in order to determine the form of structure and identify suitable sampling and testing locations.

Where required, the inspection was supplemented by covermeter testing and drilling to confirm the construction.

3.2 Concrete

Durability testing, comprising measurements of the depths of concrete carbonation and cover to the reinforcement, was carried out on selected concrete elements.

The concrete cover was measured using an electromagnetic covermeter.

The carbonation depth was measured by spraying phenolphthalein indicator solution onto freshly exposed concrete surfaces.

Drilled concrete dust samples were removed from selected accessible concrete elements.

A small number of samples were removed in three depth increments in order to identify any chloride gradient into the concrete.

The samples were analysed to determine the chloride ion content and identify the presence of HAC.

The analysis was undertaken at a UKAS accredited laboratory.

We were also provided with additional samples removed by others from the East and South elevations.

3.3 Access/Limitations

The sampling and inspection regime was designed to be representative of the concrete elements within the property but essentially represents a snap shot of the concrete in various accessible locations only.

Internal areas were agreed in advance

External samples and tests were limited to the West elevation from a 24m MEWP.

The findings relate to the areas accessed only.

4. Site Observations

4.1 Structure

- The structure was found to comprise insitu cast reinforced concrete columns and downstand beams supporting flat insitu cast reinforced concrete slabs.
- The lift shaft walls were insitu cast reinforced concrete.
- The shear walls within the stairwells were concrete block or brickwork in the areas investigated.

4.2 Concrete Condition

Internal

- The majority of the internal concrete was obscured but, where in the areas where it was exposed and inspected, typically appeared to be in a satisfactory visual condition.

External

The following concrete elements to the West elevation inspected were externally exposed:

- At ground, first and second floor levels the edge beams were exposed.
- Numerous areas of cracking and spalling were noted to the elements. This damage appeared to be due to corrosion of the steel reinforcement bars.
- These had been repaired/skimmed and rendered in places.
- Supporting concrete columns to these elements had also spalled at low level.
- Above 2nd floor level the elevation was clad with light coloured precast concrete cladding panels.
- A small number of these had spalled in places.

5. Concrete Testing

The results of the analysis carried out on the concrete samples are shown in the following table, which also indicates the site durability testing results.

Ref	Location Description	Cover to Reinforcement (mm)	Carbonation Depth (mm)	Chloride Ion Content (% by weight of cement)	Presence Of HAC
	Internal				
1	4 th Floor Column	25 - 30	10 - 15	0.42	-
2	Roof Beam	30 - 35	15 - 20	0.33	-
3	Roof Slab	20 - 30	10 - 15	0.30	Not Present
4	3 rd Floor Lift Shaft	35 - 40	5 - 10	0.34	Not Present
5	3 rd Floor Column	50 - 55	25 - 30	0.28	-
6/A	2 nd Floor Column (5 – 25mm)	-	-	0.30	-
6/B	2 nd Floor Column (25 – 50mm)	-	-	0.30	-
6/C	2 nd Floor Column (50 – 75mm)	-	-	0.44	-
7	3 rd Floor Slab	25 - 30	10 - 15	0.21	Not Present
8	2 nd – 3 rd Floor Lift Shaft Wall	20 - 25	5 - 10	0.34	-
9	1 st Floor Column	55 - 60	5 - 10	0.32	-
10/A	Ground Floor Lift Shaft Wall (5 – 25mm)	-	-	0.47	-

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10/B	Ground Floor Lift Shaft Wall (25 – 50mm)	-	-	0.41	-
10/C	Ground Floor Lift Shaft Wall (50 – 75mm)	-	-	0.33	-
	External				
11	Ground Floor Beam - Spalled	5 - 10	5 - 10	0.44	-
12/A	Ground Floor Column (5 – 25mm)	-	-	0.44	-
12/B	Ground Floor Column (25 – 50mm)	-	-	0.51	-
12/C	Ground Floor Column (50 – 75mm)	-	-	0.41	-
13	2 nd Floor Cladding Panel	-	-	-	Not Present
14	2 nd Floor Cladding Panel	-	-	-	Not Present
15	2 nd Floor Cladding Panel	-	-	-	Not Present
16	2 nd Floor Edge Beam – Spalled & Repaired	25 - 30	<5	0.46	-
17	2 nd Floor Edge Beam – Spalled & Repaired	25 - 30	<5	0.50	-
18	Ground Floor Beam	30 - 35	<5	0.53	
	Samples Provided By Others				
E1	East Elevation Cladding	-	-	-	Not Present
E2	East Elevation Cladding	-	-	-	Not Present
E3	East Elevation Cladding	-	-	-	Not Present
S1	South Elevation Cladding	-	-	0.44	-

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S2	South Elevation Column	-	-	0.71	-
S3	South Elevation Edge Beam	-	-	0.56	-
S4	South Elevation Edge Beam	-	-	0.42	-
S5	South Elevation Cladding	-	-		Not Present
S6	South Elevation Cladding	-	-	0.31	-
S8	South Elevation Cladding	-	-	-	Not Present

Notes:

1 - The chloride content has been calculated on an assumed cement content of 14%.

It is generally accepted that the maximum upper chloride limit for typical reinforced concrete is 0.4% by weight of cement (BRE Digest 444, BS 5328). Above this limit the risk of corrosion of embedded steel is increased under damp conditions.

15 of the 26 samples analysed for chlorides were found to contain elevated (medium) chloride levels.

The incremental samples did not indicate any significant chloride gradient into the concrete.

None of the other samples analysed were found to contain chloride levels that exceeded the 0.4% limit.

No evidence of HAC was identified within the concrete samples analysed.

6. Discussion & Recommendations

6.1 Reinforced Concrete Durability

Embedded steel reinforcement is protected in concrete by the alkalinity of the surrounding cement matrix. The ingress of carbon dioxide (Carbonation) gradually reduces the alkalinity of the concrete, thereby removing the protection.

Once moisture and oxygen reach the unprotected steel corrosion can occur.

The presence of chloride ions increases the potential for corrosion. The chlorides may have been included at the time of construction or may have ingressed as de-icing salts or sea spray.

Cracks or poor quality concrete will allow the ingress of carbon dioxide, moisture and oxygen. Hence corrosion will potentially be accelerated.

Therefore the durability of the concrete depends on the relationship between the cover to the reinforcement, the carbonation depth, the chloride ion content, the exposure conditions and quality of the concrete.

6.2 High Alumina Cement (HAC)

HAC was used, predominantly in precast concrete elements, due to its rapid strength gain and resistance to some chemicals.

However, when it is subjected to heat and humidity during curing, or at any time subsequently, normal calcium aluminate hydrates undergo “conversion” which is a crystal change accompanied by an increase in porosity and loss of strength.

This process was understood to have caused a number of well publicised failures during the 1970's. The use of HAC was subsequently banned.

It is now generally accepted that all HAC will be effectively converted and the conditions which give rise to conversion, and its structural performance after conversion, are now well understood.

The degree of conversion however does not directly correlate with the residual strength of the concrete. Structural testing is required to confirm the actual strength of the concrete.

6.3 Concrete Within the Property

Internal Concrete

Elevated chloride levels were identified within a number of the samples removed from the internal structural concrete.

There was no significant chloride gradient into the concrete and therefore the chlorides were probably included at the time of construction (either added at casting to speed curing in cold weather or through the use of poorly washed marine aggregate).

The concrete cover to the internal structural concrete was found to exceed the carbonation depth and therefore the durability of the internal concrete appeared to be intact.

In addition, in the areas that were accessible and visible the internal structural concrete generally appeared to be in a satisfactory visual condition.

Therefore, whilst the elevated chloride levels identified within the structural concrete could potentially initiate premature corrosion of the embedded steel reinforcement, as long as the internal concrete is kept dry this deterioration should not occur.

No HAC was identified.

External Concrete

Elevated chloride levels were identified within a number of the external concrete samples.

These were similar to the levels identified internally and therefore it would also appear that these were included at the time of construction.

These chloride levels, allied to localised areas of low concrete cover (hence the concrete being carbonated), would appear to have initiated corrosion of the steel reinforcement.

The expansive effects of the corrosion has then cracked and spalled the cover concrete. This has especially occurred to the floor slab edge beams/nibs.

Repairs have already been attempted (and failed) to these elements.

Any future external repairs will need to incorporate additional protection to minimise the possibility of future deterioration occurring.

This could include an anti-carbonation coating and possibly corrosion inhibitor.

No HAC was identified.



RICHARD SHARP CEng BSc MINDT MIAQP - Director

APPENDIX A
PHOTOGRAPHS



1 – View of the rear elevations to the property.



2 – Typical insitu cast concrete construction within the property.



3 – As above.



4 – View of a typical spalled edge beam/nib.



5 – As above.



6 – A low level external spall.



7 – A spalled edge beam/nib.



8 – As above.

APPENDIX B

CONDITIONS & LIMITATIONS

Conditions and Limitations

1. Access

If we observe evidence to suggest that concealed parts of the structure and fabric might be defective or contain hazardous materials, we will advise you accordingly and make recommendations for further investigations.

However, unless otherwise instructed by you, we will not open-up for inspection any permanently enclosed or concealed parts of the structure and fabric.

Access to some areas may be restricted or denied. If we find that our inspection has been excessively limited we will advise you accordingly and seek your further instructions.

Our report will list any significant internal and external areas that we are unable to inspect.

2. Deleterious Materials

Deleterious materials surveys are carried out by suitably experienced and qualified staff.

The sampling regime and inspection will be designed to be representative of the materials within the property. The samples taken would represent a fraction of the materials present within a property however.

In addition, the sampling and inspection locations will be dictated by accessibility, practicality and the requirement to minimise occupant disturbance.

The results provided will relate to those materials/elements tested and reported on only.

Therefore there cannot be a guarantee that all potentially deleterious materials have been identified.

3. Asbestos Surveys

Asbestos surveys are carried out by experienced P402 qualified staff in general accordance with HSE document "Asbestos: The survey guide".

The survey types available are as follows:

1. Management Survey – Sampling Survey, samples of representative suspect materials taken for laboratory analysis.
2. Refurbishment & Demolition Survey – Pre demolition/major refurbishment survey.

In addition we can undertake a pre acquisition/overview – A walk around non intrusive survey.

All reasonable efforts will be made to detect asbestos. It must be recognised however, that areas of asbestos may be in inaccessible locations or obscured.

In addition the inspection locations will be at selected or random spot positions (although designed to be representative of the property) only.

As such **no guarantee, regardless of the type of survey**, can be given that all asbestos materials will be located.

Except where specifically requested, land contamination would not be included.

RS will accept no responsibility for claims arising as a consequence of exposure of others to asbestos containing materials and accept no liability in the situation that asbestos is not identified during a survey.

4. Investigations and Testing

RS will employ techniques and equipment considered appropriate for particular investigations. Should it subsequently become clear that additional works are required this will be communicated to the client.

RS are therefore unable to guarantee that the results of each and every investigation will be completely conclusive but will recommend additional works where appropriate.

Any non-destructive testing may be limited by the capabilities of the technique/method.

The results of water ingress investigations can result in remedial works being proposed in order to eliminate certain details or identified defects.

Water can appear within a building at one location although there is more than one source. It must therefore be recognised that additional testing or remedial works may therefore be required.

5. Payment Terms

Invoices are issued either at completion of the appointment or, in the case of ongoing projects, approximately monthly.

Payment is strictly due within 28 days of the invoice date.

RS retain the right to charge an interest of 8% above standard base rate on overdue accounts.

6. Obligations

RS will exercise all reasonable skill, care and diligence in the provision of the service and act in good faith, but accept neither responsibility nor liability for any loss or damage arising from the service.

7. Retention of Title, Copyright and Samples

The Title for all reports, advice, design or material of any kind provided in writing or orally is retained until payment for all work has been made in full.

Our report shall be relied upon only by the party to whom the report is addressed and only when payment for the report has been made in full.

Copyright on all of the above mentioned is retained permanently unless expressly assigned by RS in writing.

Samples or materials may be disposed of after three months unless express instructions to the contrary have been given.

8. Costings

The costings for remedial works are estimates only based on observations made during the inspection.

They are approximate figures that indicate the general scale of likely costs only and are not corroborated by tenders from the market.

Except where stated, the costings would not include for VAT, professional fees or access provision.

APPENDIX

iv) BRE DIGEST PART 444 PART 2

Corrosion of steel in concrete

Investigation and assessment

Digest 444 Part 2

BRE Centre for Concrete Construction

This Digest is in three Parts. Part 1 examines the durability of steel in concrete. With Part 2 on investigation and assessment, and Part 3 on protection and remedial work, it sets out the basic principles for all those concerned with the design and maintenance of durable concrete structures: owners, tenants on repairing leases, architects, material scientists and contractors, but particularly surveyors and engineers involved with design, inspection and assessment, as well as with the remediation and protection of concrete structures. It also examines existing standards of construction and the lessons learned from the investigation of cases of corrosion in concrete.

This part of the Digest, Part 2, provides concise guidance on the format for investigations of corrosion of steel in concrete, the techniques employed and how this can lead to a prognosis for the future performance of existing reinforced concrete structures.

Part 1 explains the physical, chemical and electrochemical processes involved in the deterioration of reinforced concrete by corrosion.

Part 3 describes the protection and repair of concrete structures subject to corrosion damage, or which are expected to need such measures to minimise future damage or deterioration.

Digests 263, 264 and 265 are withdrawn.

Introduction

Although well designed reinforced concrete structures (a term which includes buildings, bridges, sea defences etc) generally show excellent durability, problems still arise from the corrosion of reinforcing steel in structures exposed to aggressive environments (eg motorway bridges, car parks and marine installations). Furthermore, construction practice over the past few decades has left a legacy of existing structures with a range of defects that contribute to reinforcement corrosion. These are likely to require repair and remediation, or even replacement.

Before undertaking any repairs to a reinforced concrete structure, it is essential to discover the causes of distress or deterioration. It is, of course, important to balance the cost of

any investigative work with the benefit that the information derived will provide; but an appropriate investigation and assessment strategy may be critical to the whole process of both maintaining and extending the residual service life of the structure.

Corrosion of the reinforcement is one of a number of possible mechanisms of deterioration. Other common mechanisms are:

- structural overload;
- impact and abrasion;
- long term shrinkage, creep or thermal expansion;
- other material based factors such as alkali aggregate reaction (AAR), shrinkage of aggregates, sulfate attack and freeze-thaw attack;
- problems arising during construction such as plastic shrinkage and settlement.



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An effective durability investigation must be able to distinguish between deterioration mechanisms and assess whether they may indirectly lead to reinforcement corrosion if no action is taken.

Recent improvements in design should mean that, for new structures exposed to more moderate environments, the main relevance of an investigation and assessment of reinforcement corrosion will be in the evaluation of in-service performance to provide confidence in future behaviour.

The aim of this second Part of the Digest is to provide concise guidance on the format for investigations of corrosion of steel in concrete and the techniques employed, and how this can lead to a prognosis for the future performance of existing reinforced concrete structures. In this context the objective will be the rectification of corrosion damage rather than structural enhancement. Some aspects of structural enhancement are covered in the later section, 'Structural implications of reinforcement corrosion'.

Reasons for assessing reinforced concrete structures

Reinforced concrete structures can be conveniently divided into two main categories on the basis of the way they are managed.

- Structures falling within an asset management system and therefore tending to be subjected to an ordered approach to inspection and assessment (eg bridges, industrial plants and similar facilities).
- Structures not generally part of an organised system, so inspection and assessment tends to be reactive to a problem or undertaken on an ad hoc basis, possibly due to a change in ownership. Many structures tend to be managed this way.

Structures in asset management systems

Bridges and their management illustrate the general approach that might be adopted for reinforced concrete structures within an asset management system.

The frequency and scope of inspections would be influenced by a number of factors including the importance of the structure, potential risk and consequence of failure and its environment.

A systematic approach might involve a combination of **superficial**, **regular** and **as-required** inspections, with as-required

inspections often being instigated in response to findings made during earlier stages.

Appropriately trained and experienced staff would normally make these inspections. In many cases, circumstances arise which enable incidental inspections to be made by others. Although they may not have any formal training, their more frequent observations can be helpful in keeping a watchful eye on the structure, potentially allowing early detection of obvious problems.

Superficial inspections

When personnel visit a structure for any reason, it provides an opportunity for observations of the general condition of the structure to be made, noting any visible indications of potential durability problems and maintenance issues (eg spalled concrete, leaking joints and ineffective drainage).

Regular inspections

General inspections

Typically every two to five years. While much of an inspection will be undertaken from locations of easy access, it would be expected to include those parts not generally visible or accessible.

Benchmark inspections

Every 20 to 25 years or when responsibility for, or the function of, a structure changes. A benchmark inspection should be carried out in sufficient detail to establish any significant alteration in the form or condition of the structure.

Inspections on an as-required basis

Particular inspections

Detailed inspections of particular parts of a structure at programmed intervals, according to a schedule reflecting its form of construction, environment, condition and materials, and the maintenance history of particular components.

Special inspections

Similar in scope to particular inspections, but undertaken for a specific reason rather than at programmed intervals.

In the case of bridges, most owners follow an inspection regime based on specifications by the Highways Agency. Further details are provided in Concrete Bridge Development Group Technical Guide No 2^[1].

Ad hoc investigation and assessment of structures

If not part of an asset management system, the need for investigation will often be driven by factors such as the short term needs of the structure owner or the condition of the structure. A significant proportion of investigations are of a diagnostic nature in response to an observed or reported defect such as cracking or spalling of concrete. It is recommended that initial visual inspections are carried out before testing as these can provide information on defects or deformations. The priority, after an initial inspection and before

testing, is to assess the magnitude of the problem and determine the primary cause. It must be established early on whether or not the problem is corrosion related or whether corrosion has played a part in the overall deterioration. Only then can an investigation to determine the level of deterioration due to reinforcement corrosion begin.

The decision process for investigating and assessing structures is summarised in Figure 1.

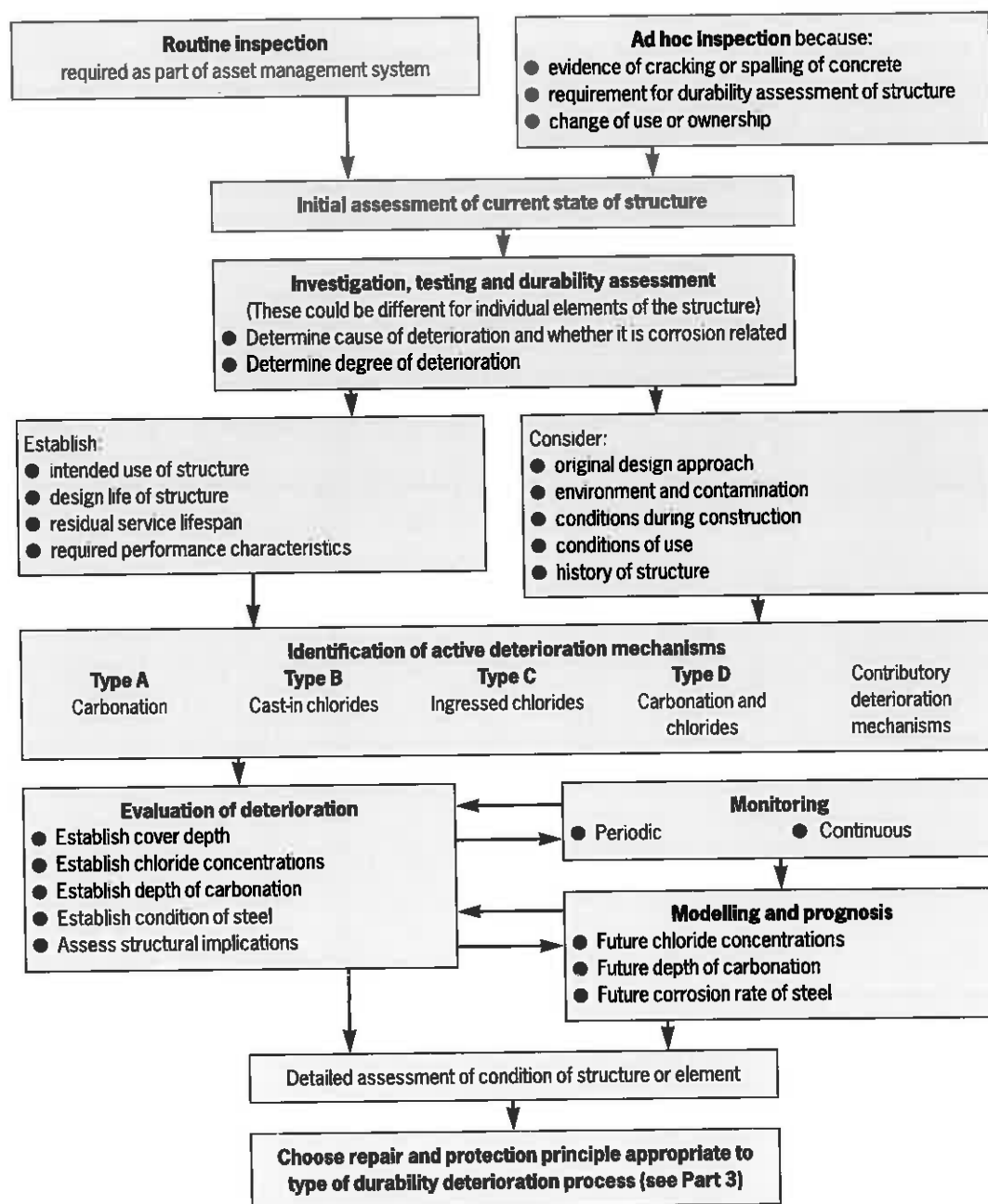


Figure 1 A summary of the decision making process for investigating and assessing corrosion in reinforced concrete

Testing

Usually testing will be part of an investigation and may include simple on-site testing (eg estimation of depth of carbonation), non-destructive testing (NDT, eg half-cell potential mapping) and the testing of material samples (eg determination of chloride concentrations and petrographic examination).

Testing undertaken to estimate potential expenditure on a repair and maintenance project must be commensurate with the likely value of the project. In some cases testing will not be required. However elimination of testing for the wrong reasons could prove to be an expensive error. (Testing is discussed in later sections of this Part.)

Monitoring

Monitoring is primarily a diagnostic or control process to help understand the in-service performance or management of a structure. It is also a valuable tool in the routine assessment of a structure. Monitoring may be either a periodic or repeated activity, or a continuous recording of data. Electrochemical monitoring is discussed in more detail in Digest 434, but it is also referred to in later sections on testing in this Part of the Digest.

The ability to monitor the condition of steel reinforcement in concrete is useful in three ways. First of all, owners can obtain information on the condition of their structures and determine whether there is currently a problem with corrosion. Secondly, monitoring can be used by owners to assess the effectiveness of repairs carried out on a structure. Thirdly, some monitoring techniques should enable predictions to be made as to the likelihood of future problems.

Site inspections

Two assumptions are made about successful site inspections.

- Trained and experienced personnel are used.
- These personnel can access comprehensive and detailed plans and checklists.

There are a few obvious and simple techniques that should be carried out initially to obtain an overall picture of the condition of a reinforced structure. A good visual survey is probably the most important step at this stage. Inspection should reveal tell-tale signs such as cracking of the concrete, rust staining and delamination; but perhaps more importantly, signs of water retention, inadequate drainage and, particularly in flat surfaces such as car parks, formation of

puddles. Edges and crevices should be studied carefully. An assessment of possible environmental factors – exposure of the structure to direct rainfall, sun and wind, for instance – should be made. Key parts of the structure, such as movement joints, should be identified and assessed.

Guidance on an investigative approach cannot be too prescriptive as every structure is different and local conditions will vary. It is unlikely that any of the methods documented, taken individually, will provide definitive information about a potential problem, and it will often be necessary to use a number of inspection techniques and procedures to obtain a complete picture.

The inspection process

The procedures for performing inspections are well established, although the policies of different structure owners may have a bearing on the nature and frequency of inspections.

The process of inspection and assessment will generally fit within a basic framework, although the scope of this may vary depending on the importance, size and complexity of the structure. A large investigation might progress in the way indicated below, with the work broken down into a number of phases:

- establish a brief;
- consider health and safety issues;
- carry out desk studies and a first appraisal;
- carry out an initial site reconnaissance and inspection;
- review the resulting data;
- examine each problem in detail, if necessary by further and fuller inspections;
- analyse and interpret all data;
- establish the cause of each problem;
- propose remedial options;
- prepare a report.

In less demanding circumstances a simplified version of the above may be acceptable.

The requirements for an investigation will vary considerably. Accordingly, it is important to establish and record clearly from the outset its true purpose before any work commences; in other words, to establish a clear brief so that both the client, consultant and contractor are aware of exactly what is required to be undertaken and delivered. As part of any desk studies, it will be essential to determine in as much detail as possible the full history of the structure and to review any reports previously carried out.

In some cases, investigations may involve revisiting structures, or elements of structures, that have been already investigated and repaired, and reviewing their condition. These visits should be made part of the initial brief.

Typically testing will not be considered until desk studies of available information and a preliminary visual inspection of the structure have been undertaken. These stages in an investigation should establish the objectives for testing and how the information obtained might be expected to contribute to the overall decision making process.

Testing that is targeted

Testing is more effective if it is targeted at what are anticipated to be critical parameters and particular locations, rather than adopting a widespread and random approach to enable general conclusions to be drawn by means of statistical analysis. The target approach assumes that test locations can be determined in advance, that appropriately experienced personnel will carry out the work, and that they will have access to knowledge about previous work and investigations on the structure and its service history. This is usually gained from the desk studies and initial inspection. Specialist expertise may be needed when planning a testing programme.

Testing regimes are often developed in a phased or incremental manner so that the planning of later phases of an investigation can benefit from the findings of earlier inspection and assessment work. Because testing is usually an expensive process, it is necessary to target carefully the types of testing undertaken and the number of results sought and to focus upon those locations which might be critical. It is essential that the information required from the testing regime is clearly identified at the outset. Consideration needs to be given to:

- the extent of possible variations of the properties of the concrete placed in relation to individual members used throughout the structure, as well as in the environment to which it has been exposed, and the possible significance and implications of this;
- the limitations of the testing and inspection procedures available;
- difficulties associated with undertaking the selected tests, their reliability and variability;
- equipment calibration requirements, particularly where on-site calibration is necessary;
- interpretation of the test results and any problems which this may present.

Initial site reconnaissance

Aims of an initial inspection

As part of a routine or diagnostic investigation, the purpose of an initial reconnaissance phase is to identify the presence and then determine the likely cause of any deterioration to a reinforced concrete structure. In some cases this will be sufficient to provide the requisite information and a detailed investigation may not be required.

During the same investigation, some determination of depth of carbonation and sampling for chloride content may be carried out to provide a preliminary indication of the extent of any problem, and to form the basis for a more detailed investigation.

Deterioration generally manifests itself as cracking of the concrete, and the shape, pattern and location of cracks along a reinforced element are frequently a good indication of the likely cause. Figure 2 provides guidance on the possible causes of cracking in reinforced concrete. Cracking of concrete structures acting in flexure (eg beams and slabs), though, is an expected aspect of their behaviour, and consideration of cracking forms part of the structural and durability design process.

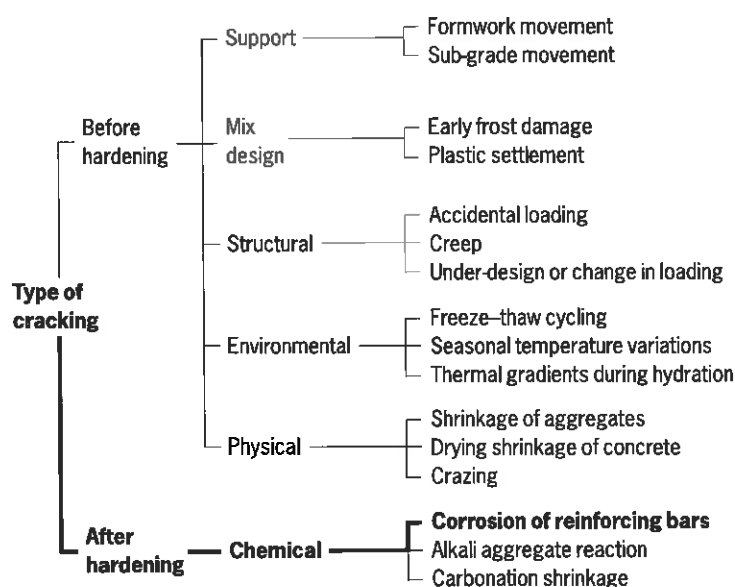


Figure 2 Types of cracking in reinforced concrete and their causes

Testing as part of an initial inspection

The initial inspection is aimed at determining whether a problem exists on site and, if so, the likely cause of the problem.

The focus of the tests is likely to be different for different types of structures; for example, carbonation of cover concrete is not usually a problem in prestressed concrete structures using high performance concrete because of the high strength and quality of the concrete mix employed. Chloride ingress is more likely to pose a threat to a car park or a structure in a marine environment than a building at an inland location.

An early investigation may combine a range of simple on-site tests including visual examination, a covermeter survey, and depth of carbonation and chloride analysis. Where necessary these combined results can be used to map out a more detailed investigation where the range and complexity of testing may be increased.

It may be necessary to examine the variability in performance of a structure with regard to observed or potential problems of corrosion. Similar elements in a structure could perform differently due to factors such as orientation and local environment.

Detailed examination

Aims

Following the initial phase, a more detailed examination may be required. This may benefit from the earlier work by using the results to target the critical areas of the structure. The critical areas would be considered as those with the highest risk of deterioration or with the most significant consequences, should deterioration occur.

In addition to simple on-site tests similar to those taken during the initial inspection, a detailed examination will often require a range of NDT methods and selected testing of material samples. The latter will typically involve the coring or drilling of samples from the structure for further detailed analysis. This may need the services of a specialist contractor.

Testing as part of a detailed examination

In many instances the problem is clearly defined and the selection of the area for investigation is relatively straightforward. Depending on the brief it may also be necessary to widen the scope of the examination to demonstrate whether the problem detected in the initial inspection is a local one or widespread within the structure.

An aspect of a more detailed investigation could involve assessing the condition of the

concrete providing the protection to the embedded reinforcement. Tests to assess concrete quality could be specified and include compressive strength, petrography, permeability and cement content.

Guidance on the selection of tests will come from the initial inspection, and it may be that the same tests are required but in greater numbers targeted at particular areas to give greater statistical confidence in the results.

A suitable rate of sampling for screening for cast-in calcium chlorides is one concrete sample from 10% of each type of component within a structure, or 10 samples for each type, whichever is the greater. In structures containing fewer than 10 components, sampling is not appropriate and therefore each component should be tested. In structures where a 10% sampling rate is difficult to achieve, it has to be accepted that the risk of not detecting components containing additions of chlorides will increase. In these circumstances it may be possible to maximise the effectiveness of the sampling regime by taking into consideration other factors which may indicate the use of calcium chloride. However, concrete components containing calcium chloride additions may be distributed at random. In these circumstances it may be appropriate to perform a quick on-site test on a small dust sample obtained from each component to provide an early rough positive or negative answer. More detailed and selective testing can then follow.

Whatever sampling regime for chloride screening is chosen, the objective should be to sample as many different types of components at different locations as possible, giving priority to locations where it may be reasonable to suspect the use of calcium chloride, or where the effects of added or ingressed chloride will be more significant.

Health and safety

When planning an inspection, account should be taken of the nature of the structure and its access, and the appropriate equipment and clothing which may be necessary. If work is carried by one person, they should report in at specified intervals.

Dilapidated structures or confined spaces may constitute particular hazards. Before starting inspection or testing, it is wise to make a general assessment of the state of the structure and only when satisfied that it is reasonably safe proceed with the proposed activity. Further guidance is given by the Institution of Structural Engineers in Appendix 2 of *The appraisal of existing structures*^[2].

Testing

Investigations are rarely exactly the same, so it is not possible to prescribe a combination of tests for a particular situation. Table 1 gives some guidance for selecting tests (the list is not exhaustive) commonly used in investigating corrosion in reinforced concrete. These tests include both site and laboratory techniques, and are described in the sections that follow.

Further testing could determine:

- concrete strength and surface hardness (eg compressive strength of cores and rebound hammer testing);
- comparative quality and uniformity of concrete (eg ultrasonic pulse velocity and radiography);
- presence of alkali aggregate reaction (eg petrography and expansion testing);
- surface permeability and absorption of concrete (eg ISAT and water or gas permeability testing).

It may be prudent also to run checks on the distribution and structural performance of steel reinforcement including the presence of links, the condition of movement joints, the degree of structural redundancy present etc. These checks would assist in understanding the structure in sufficient detail to determine its sensitivity to the effects of deterioration, and would be essential if structural weakening is suspected.

Additional information on selected tests

In some structures the need for investigation may not extend beyond some limited testing. Experienced practitioners might start their investigations using some of these tests; others might consider a visual survey more appropriate as a first step.

Where deterioration is believed to be the result of corrosion of the reinforcement, a guide to the investigation work required is provided in Figure 3.

Visual inspection

While visual inspection alone will not usually be sufficient to determine whether steel reinforcement is corroding, it is, nonetheless, a powerful procedure when undertaken by a suitably experienced person. Where problems are identified, it is useful to record defects and the locations where test procedures are to be performed. Relevant indicators and defects which might be observed include:

- cracking;
- scaling – local flaking or peeling of the surface portion of hardened concrete or mortar;
- spalling – concrete fragments, usually detached from the body of the concrete;
- lime staining on the concrete surface;
- efflorescence – deposition of white salts;
- rust staining – brown or rust coloured stains;
- weathering – changes in properties, such as colour, texture or strength;
- honeycombing – clear evidence of voids or spaces between coarse aggregate particles;
- dampness – wet or moist areas of concrete, or ponding;
- leakage through joints – generally water, but possibly contaminated with chlorides or other substances;
- condition of previous repairs;
- abrasion – progressive loss of mass from the concrete surface;
- physical damage, such as vehicle impact;
- delamination – detected by hammer or chain drag;
- exposure to inclement weather conditions – rainfall, wind etc.

Table 1 Simplified guide to aid selection of testing

Information sought	Diagnostic testing or inspection technique	Parameter selection *
Extent and significance of cracking and visible defects	Visual inspection; photographic records	1
Presence of reinforcement and depth of cover	Covermeter	1
	Sub-surface radar	3
	Physical exposure by excavation or cored sample	2
Depth of carbonation	Phenolphthalein test (not applicable to HAC)	1
	Petrographic or microscopic analysis	2
Presence of chlorides or chloride profiles.	Drilled, lump or cored samples for presence of contaminant by laboratory analysis; incremental sampling required for profile determination	1
Presence of sulfates	Site chemical tests (HACH, QUANTAB etc)	3
	performed on drillings	3
Delamination	Sounding survey (tapping, chain drag etc)	1
	Examination of cored samples	2
	Thermography	3
	Impact echo test	3
	Ultrasonic pulse velocity	3
	Sub-surface radar	3
Reinforcement corrosion. Condition of embedded steel reinforcement and prestressing tendons	Half-cell potential surveying	1
	Resistivity evaluation	2
	Physical exposure by excavation or coring	2
	Impedance spectroscopy	3
Corrosion rate	Linear polarisation resistance	2
	Galvanic current measurement	3
Moisture content	Drilled or lump samples	1
	Resistance/capacitance/dew-point probes	2
	Sub-surface radar	3
	Thermography	3

* 1 = essential, 2 = desirable but not essential, 3 = may be helpful in some cases

In corrosion affected concrete, there may be no outward signs such as rust staining, spalling or cracking. Visible evidence of damage depends on the corrosion process, the influence of chlorides, the quality of the concrete and other factors. A true appreciation of the problem may be gained only by removing the concrete cover and exposing the steel for examination. A covermeter survey might be useful at this initial stage to locate the reinforcement and establish its relationship to any visible damage.

Once the cover has been removed in an area of spalled or cracked concrete, the type of rust present should be recorded as it may indicate the cause of the problem. Carbonation usually results in widespread corrosion of the steel and usually produces flaking layers of rust over long lengths of the bar. High chloride content, on the other hand, generally leads to short lengths of more severe corrosion with uncorroded bar on either side (or on the side furthest from the surface if the chloride source was external), particularly where the steel has pitted.

An examination should also be made to determine whether there has been any loss of steel cross-section due to the corrosion of the bar. Significant loss of steel may have structural implications that need to be addressed (see the later section, 'Structural implications of reinforcement corrosion').

Following the initial visual inspection, further testing options can be considered.

Depth of carbonation

The depth of carbonation in concrete can be readily assessed on a suitably oriented section of concrete with a freshly broken surface perpendicular to the surface of the component. Site or laboratory measurement of carbonation depth for Portland cement (PC) concrete is dealt with in Digest 405; similarly, IP 11/98 deals with HAC concrete. The fracture surface is sprayed with an indicator solution, usually phenolphthalein in a mixture of water and ethyl alcohol. The rapid development of a pink colouration indicates where the concrete has not carbonated and is still capable of protecting the steel from corrosion in the absence of chloride. Other indicators (eg bromocresol purple and m-cresol purple) should normally be used for HAC concrete.

While the phenolphthalein test is generally reliable for PC concrete, there are situations where identification of the carbonation layer is unclear. It is then necessary to confirm these indications by petrography. Instances where phenolphthalein is unreliable are when the colour change to pink does not occur rapidly (within approximately 30 seconds) or where the development of the pink colouration spreads with time.

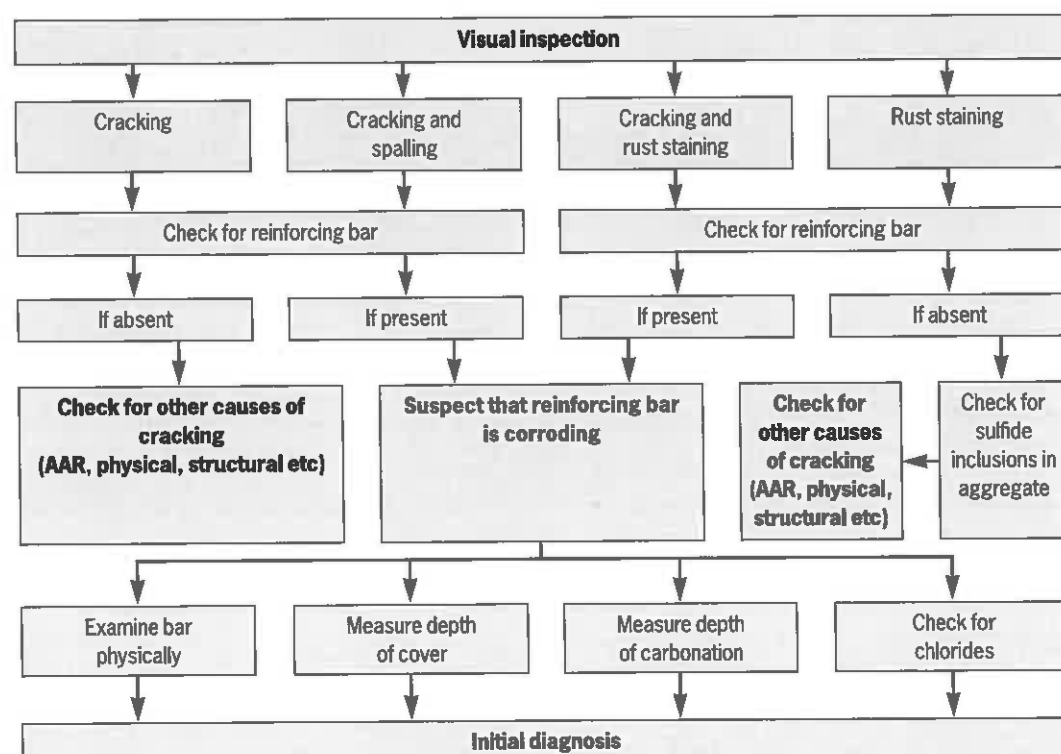


Figure 3 Preliminary diagnosis of reinforcement corrosion

Chloride ion content

Determining the chloride content of concrete is an important part of any investigation of the deterioration of reinforced concrete. Sufficient quantities of chloride ion present in concrete can stimulate reinforcement corrosion, even in highly alkaline conditions. The main risk is from chloride, either introduced before the setting of the concrete or subsequently during the service life of a structure (eg in a marine environment or via de-icing salts). Prior to setting, chlorides may have been introduced as part of the mix ingredients or, in pre-1977 concrete components, as the set accelerator calcium chloride.

The method by which dust samples can be collected by drilling holes into the concrete is described in IP 21/86. Powder samples obtained by percussive drilling or from lump samples of concrete can be sent for laboratory analysis to determine the presence of chlorides (and sulfates). When a structure is thought to be exposed to chlorides (eg in a marine environment or arising from de-icing salts), and it is necessary to establish chloride penetration profiles at selected locations on a structure, samples must be obtained from different depths below the surface of the concrete. This can be achieved by either incremental percussive drilling, or from cored samples which are subsequently cut into sections and ground up for laboratory chemical analysis. Chemical analysis will provide the chloride ion content as a percentage, either by weight of sample or by weight of cement. If a specific request is not made to determine the cement content of the samples provided, a figure for cement content will usually be assumed.

The sampling regime must be carefully designed for the particular structure and the nature of the envisaged contamination (ie introduced at the time of construction or subsequently by ingress). Site determinations provide the flexibility to amend the sampling regime in the light of the results obtained.

Covermeter surveys

The depth of cover concrete, and the size and location of reinforcement, can be estimated using an electromagnetic covermeter. Although the equipment is straightforward to use, considerable care and skill can be required to obtain valid results. In heavily reinforced members it may not be possible to obtain reliable results. The performance of some equipment may be affected by low temperatures (<0 °C) and by particles of magnetite which may be present in mixes

containing pozzolans (especially fly ashes), sands and crushed rock. Some modern equipment facilitates data collection by connection to a data logger. Covermeter surveys need to be correlated with other information from the investigation, including defect mapping, carbonation depths and chloride profiles.

Half-cell potential surveys

The measurement of half-cell potentials is the most widely used method for assessing the likelihood of the corrosion of steel reinforcement in concrete. Although the method provides an indication of localised zones of potential corrosion activity, it does not determine corrosion rates or the degree of corrosion that has occurred. The interpretation of half-cell potentials with respect to corrosion is covered by various publications including Digest 434, ASTM C876-87 and TRL Application Guide 9^[3].

Linear polarisation resistance measurements

Polarisation resistance measurements provide an indication of the rate of corrosion of metal embedded in concrete. A major difficulty with measurements made from the surface of the concrete is defining the area of embedded metal (reinforcement) which is participating in the measurement. One approach adopted to overcome this is to confine the measurement to a known area by the use of a guard ring electrode. Digest 434 discusses the use of this technique in more detail.

Resistivity measurements

Concrete resistivity is a measure of the ability of the concrete to act as an electrolyte and carry corrosion currents. It is usually used to investigate areas of greatest corrosion risk identified by electrical potential mapping. The measurements give an indication of the rate of corrosion where the reinforcement is no longer passive and, when used in conjunction with a half-cell potential survey, its likely extent. The technique does not give any guide as to the degree of corrosion damage that has taken place. Digest 434 discusses the use of this technique in more detail.

Subsurface radar surveys

High frequency (1–3 GHz) electromagnetic impulses are transmitted from an antenna traversed over the surface under investigation. Reflections are received from internal features and discrete boundaries between materials having different electrical and dielectric properties. The technique is completely non-destructive, although exploratory holes are required to physically identify anomalies and features of interest. The resolution and penetration achieved depends upon the frequency of radiation employed, which is controlled by the antenna being used. Moisture and the presence of chlorides affect behaviour and performance. In favourable circumstances, radar can also provide some broad indications on a number of other parameters (information on density, delamination or the presence of chlorides and moisture). The engineer should seek objective advice from a radar specialist as to the ability of radar to assist in a particular situation. The radar specialist should be able to advise on the limitations of the equipment and potential difficulties in the interpretation of results in the circumstances envisaged. The BRE Report, *Application of subsurface radar as an investigative technique*, discusses this technique in more detail.

Monitoring

Monitoring may be applied to steel reinforcement in concrete and to the structure as a whole; but, while the two are interrelated, this Digest concentrates on the former.

Monitoring is primarily a diagnostic or control process used to assist in understanding the in-service performance of a structure (or an element of a structure) or in its management. Although monitoring may be supplemented by various short term investigations and testing activities, it is essentially a means of recording changes with time.

As monitoring may be carried out for a long time and involve significant expenditure, careful consideration should be given to the purpose and benefits to be gained. There needs to be a clear understanding of how the data obtained from monitoring will aid decision making and management process and the timescale over which takes place.

Defining the purpose of the monitoring is crucial as it establishes what is to be done and why, as well as how the information collected is going to be processed and interpreted. The monitoring objectives strongly influence the scope, form, frequency and duration of data collection and treatment.

Monitoring schemes

In essence a monitoring scheme to investigate problems of steel reinforcement corrosion in concrete will need to provide a method of undertaking measurements or gathering data on some predetermined basis. A range of methods is available and some of these are referred to in Digest 434. They can be separated into two main categories.

Repeat monitoring. Manual surveys, using portable equipment where necessary, including visual assessment, chloride concentration determinations and electrochemical techniques that may be influenced by the season (eg corrosion rate and resistivity).

Continuous monitoring. Electrochemical techniques involving the use of probes applied from the surfaces of existing concrete structures, embedded in new structures or retrofitted into existing structures. Continuous monitoring of data will show periodic fluctuations (eg daily and seasonal), and it is important that these are understood and not misinterpreted.

The major advantages of continuous monitoring are that:

- electrochemical probes embedded in a new structure can give early warning of potential durability problems, especially in critical areas;
- once installed, access is not required again;
- retrofitted probes can be used to assess the effectiveness of remedial techniques.

Information collected from surface, embedded or retrofitted probes is valuable in establishing the performance of materials and in providing feedback on the design process.

Prognosis

Part 1 of this Digest highlighted carbonation and chlorides (either cast-in or ingressed) as the principal factors behind the corrosion of steel reinforcement in concrete.

Clearly, if the carbonation front or a sufficiently high concentration of chloride ions reaches the steel reinforcement, it is likely that corrosion will occur. In cases where the area around the steel remains free of chloride contamination and carbonation, predicting the likely risk of corrosion in the future based on current survey results is more difficult. The following criteria can help predict the onset of corrosion:

- average and maximum depth of carbonation;
- chloride content (% by mass of cement) at the depth of the reinforcement;
- source of chloride;
- depth of concrete cover;
- environment conditions;
- age of the structure.

Interpretation of the above information should recognise when surfaces are painted or unpainted, and whether measurements are taken on the soffits or the sides of components.

Carbonation

Carbonation is assumed to follow a parabolic ingress rate:

$$d = k t^n$$

where d = carbonation depth,
 k = a constant,
 t = time,
 n = an exponent lower than 1, often taken as 0.5.

The rate constant, k , depends on a number of factors including:

- cement type and content;
- water:cement ratio;
- aggregate type;
- duration, relative humidity and temperature during a controlled curing period;
- degree of compaction;
- environmental conditions including relative humidity, temperature and the local concentration of carbon dioxide.

Fick's Laws of Diffusion

Diffusion is the process by which matter is transported from one part of a system to another as a result of random molecular motions. For isotropic substances under steady state conditions, the rate of transfer per unit area of section, J , is given, according to Fick's First Law of Diffusion, by the equation:

$$J = -D \frac{\partial C}{\partial x} \quad (\text{Equation 1})$$

where C is the concentration of diffusing substance and x is the space coordinate measured normal to the section. The constant, D , is the diffusion coefficient.

Chloride ion diffusion in concrete, however, is generally not a steady state process and the differential equation of diffusion, known as Fick's Second Law of Diffusion, is applicable.

$$\frac{\partial C}{\partial t} = D \frac{\partial^2 C}{\partial x^2} \quad (\text{Equation 2})$$

A solution of Fick's Second Law into a semi-infinite medium with the choice of appropriate boundary conditions, gives the following:

$$C_x - C_b = (C_s - C_b) \left(1 - \operatorname{erf} \frac{x}{2\sqrt{Dt}} \right) \quad (\text{Equation 3})$$

where C_x = chloride at depth x ,
 C_s = chloride content at surface,
 C_b = background chloride content (from the mix ingredients),
 D = effective chloride diffusion coefficient,
 t = exposure period;
 erf is the error function.

Chlorides

The prediction of future chloride induced corrosion is more difficult than a similar evaluation where carbonation is the principal mechanism of deterioration.

At its simplest, the risk of deterioration must be assessed in terms of whether the chlorides are part of the original concrete ingredients (cast-in) or introduced after the placing of the concrete (ingressed). Calcium chloride accelerators were banned in UK construction in 1977, although chloride levels in excess of those allowed in Standards might still be introduced accidentally as a contaminant of the mix ingredients.

Chloride ingress can be modelled as a diffusion process using Fick's Second Law of Diffusion (see box below), although other mechanisms (eg capillary suction, absorption, hydrostatic pressure, and ingress through cracks and joints) may also occur. Capillary suction and absorption are particularly significant in chloride contamination where there is prolonged drying of the concrete before exposure to chlorides.

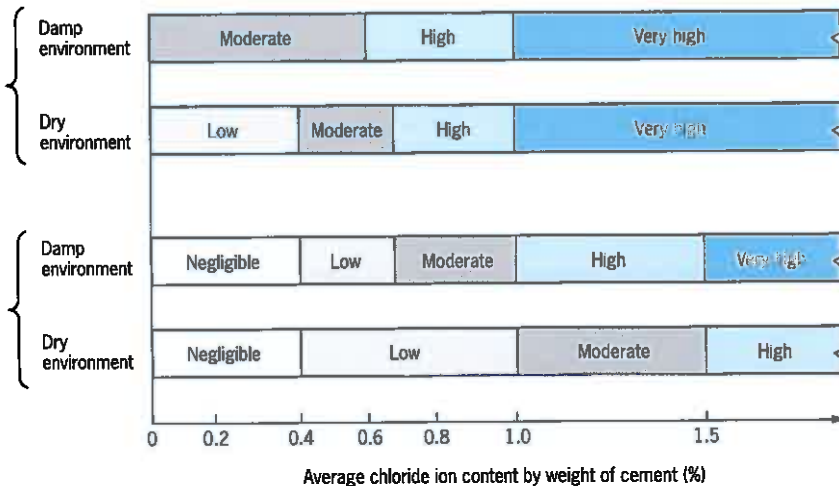
The diffusion coefficient is a property of the concrete while the surface chloride level depends on the external environment, and the cement type and content. These can be measured for an existing structure and used in Equation 3 to predict the remaining time before the chloride concentration at the reinforcement exceeds certain threshold limits, signalling the onset of corrosion. They can also be estimated from laboratory concrete, and related to controlled parameters such as the mix design and curing regime.

Risk of corrosion relative to carbonation, chloride content and environment

Once steel has been depassivated, either through the chloride level at the reinforcement exceeding a certain threshold level or the pH falling through carbonation, corrosion can initiate. There is then a propagation period during which corrosion proceeds until the serviceability of the structure is affected. The duration of this propagation period depends on the rate of reinforcement corrosion, the properties of the concrete itself (particularly its ability to accommodate the products of the expansive corrosion process), and the definition for the end of service life. Looking at the steel reinforcement in chloride laden concrete at a particular instance would only reveal whether or not the corrosion initiation stage has been reached. If corrosion is present, it is not possible to know how long it had been occurring with any certainty.

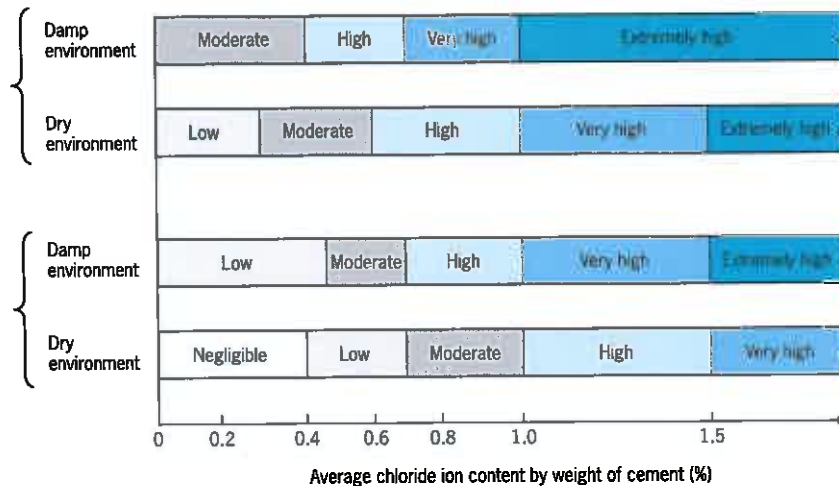
4a 25-year-old concrete structures

Where the reinforcement is in lower alkaline condition ($\text{pH} < 10$) usually as a result of carbonation, ie where the depth of carbonation has just passed the reinforcement



4b 40-year-old concrete structures

Where the reinforcement is in lower alkaline condition ($\text{pH} < 10$) usually as a result of carbonation, ie where the depth of carbonation has just passed the reinforcement



4c 60-year-old concrete structures (extrapolated data)

Where the reinforcement is in lower alkaline condition ($\text{pH} < 10$) usually as a result of carbonation, ie where the depth of carbonation has just passed the reinforcement

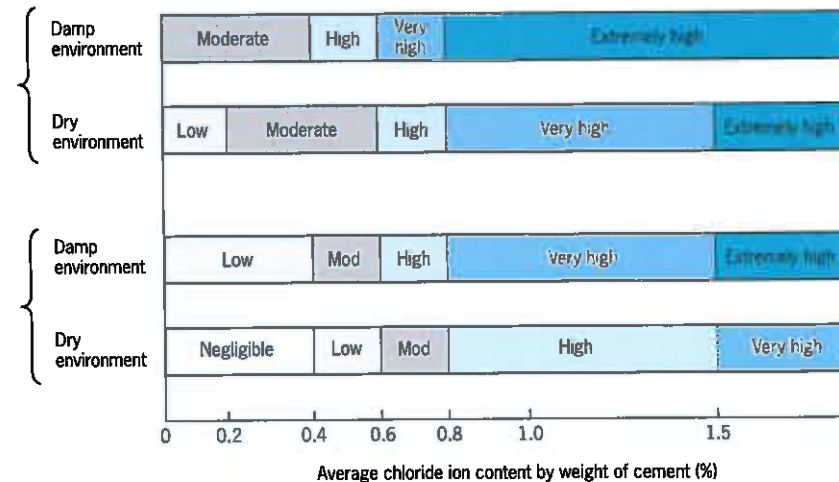


Figure 4 Estimated risk of steel reinforcement corrosion associated with carbonation, cast-in chloride content and environmental conditions

Similarly, the minimum total chloride content of concrete, the threshold level, required for corrosion to occur cannot be represented by a single value. Apart from the difficulties highlighted above, the threshold level can be influenced by other factors such as:

- water:cement ratio;
- cement type;
- pH of the pore solution;
- capacity of the cement matrix to bind chloride ions;
- whether the chloride ions are cast-in or ingressing from an external source;
- exposure conditions.

Measurement difficulties and local variations in chloride concentration, especially close to the reinforcement, introduce more variables. These limitations mean that the prediction of the chloride threshold level is very difficult. It is currently best viewed in probabilistic terms with the risk of corrosion increasing with the chloride ion concentration at the reinforcement.

Cast-in chlorides

Currie and Robery, in the BRE Report, *Repair and maintenance of reinforced concrete*, recognising that the incidence of corrosion is probabilistic and that the risk of corrosion increases with time, have attempted to give some guidance for cast-in chlorides in concrete structures representing three

different age groups: 25, 40 and a projection for 60 years old. Based on the chloride ion content (% by weight of cement), the risk of corrosion is estimated in relation to the environment (damp or dry) and the alkalinity of the concrete (carbonated or uncarbonated). Their classification of the risk of corrosion in these conditions is shown in Figure 4.

Ingressed chlorides

In the case of cast-in chlorides, there is likely to be a relatively constant concentration of chlorides throughout the initiation stage. With ingressed chlorides, however, there is a period when the level of chloride around the steel reinforcement is below the threshold level and would present no risk for corrosion of the reinforcement. Once the threshold is exceeded, the risk of corrosion initiation would depend on chloride content around the steel, which may continue to increase with time, as well as on concrete quality and environmental factors. Even under controlled conditions, for any particular chloride level above the threshold, the risk of corrosion initiation is probabilistic in nature, rising with increasing chloride concentration^[4-6]. This is reflected in Figure 5, which shows the probability of corrosion initiation from ingressing chlorides in uncarbonated concrete. It is based only on limited data^[4-6], and may have to be modified with increasing knowledge and experience. Four categories ('negligible' through to 'extremely high') have been used to simplify the interpretation process and aid the selection of remedial measures.

Further limitations of the models

In cases both of cast-in and ingressed chlorides, especially the latter, underestimating the degree of localised pitting corrosion is of particular concern because of the associated loss in cross-sectional area of reinforcement. This can be very severe in some circumstances. Part 1 discusses some of the electrochemical aspects of pitting corrosion, while the structural implications are considered in the section, 'Structural implications of reinforcement corrosion', which follows.

Carbonation can both reduce the threshold level for corrosion initiation and increase the probability of corrosion for a particular chloride concentration substantially by reducing the pH and the chloride binding capacity of the cement paste. Furthermore, when the carbonation front reaches the reinforcement, corrosion can commence in the absence of chlorides because the protective oxide layer breaks down.

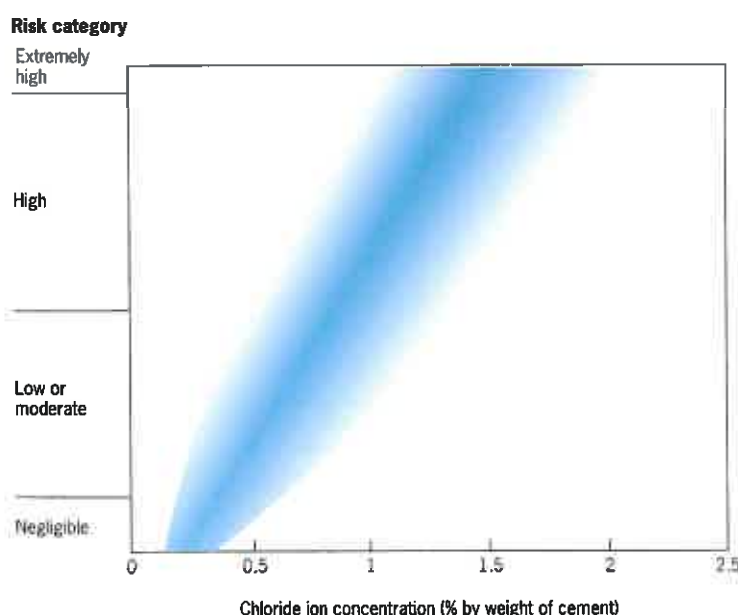


Figure 5 Estimated risk of steel reinforcement corrosion associated with ingressed chloride in the absence of carbonation

Interpretation of corrosion risk

Having made an assessment of a structure, and allocated the structure, or a component, to a risk category, an initial interpretation of corrosion risk and prognosis can be made; further guidance is given in Figure 6. This information should then be used in conjunction with Part 3 of the Digest to determine a repair strategy. The information presented should be regarded as a guide rather than a definitive prediction.

Structural implications of reinforcement corrosion

It is often necessary to consider the strength of a structure (or component) before devising a repair and maintenance strategy. This must be done by a structural engineer using the procedures outlined in the Institution of Structural Engineers' guidance document *The appraisal of existing of structures* [2]. The process is made simpler if as-built drawings and the

original design calculations are available.

The implications for safety and serviceability in concrete structures with reinforcement corrosion are best assessed by considering the effects of the attack at three levels.

Direct effect of corrosion on the performance of reinforcement

General corrosion which appears uniform along a length of reinforcement has two effects: it reduces the cross-sectional area of the steel and it creates local discontinuities in the steel surface, possibly affecting bond. These effects can reduce the tensile capacity of the steel in proportion to the loss of its cross-sectional area and may reduce the steel's resistance to fatigue damage.

Where concrete has been carbonated to the depth of the steel reinforcement and a small but uniform amount of moisture is present, the steel is likely to corrode fairly uniformly. This deterioration is often indicated by fine hairline

6a Cast-in chlorides

Risk category		Description	Possible action**			
Obtain risk category from Figure 4	Negligible	No corrosion expected	Do nothing but monitor			Feed information into model to facilitate prognosis Choose appropriate remediation process as detailed in Part 3 of Digest
	Low	With normal maintenance no significant corrosion likely to occur. Some minor corrosion may be identified				
	Moderate	Some corrosion likely to occur. Rate of corrosion likely to be slow	Determine level of corrosion and deterioration by planned inspection appropriate to risk	Regular monitoring over time	Check present structural performance	
	High	Significant corrosion likely, particularly towards the end of the selected age				
	Very high	Significant corrosion likely over considerable area				
	Extremely high	Severe corrosion inevitable. Significant area likely to be affected				

6b Ingressed chlorides

Risk category		Description	Possible action**			
Obtain risk category from Figure 5	Negligible	Little or no risk of corrosion under current conditions over the lifetime of the structure*	Do nothing but monitor			Feed information into model to facilitate prognosis Choose appropriate remediation process as detailed in Part 3 of Digest
	Low or moderate	Some corrosion possible under current conditions. Rate of corrosion likely to be low*	Determine level of corrosion and deterioration by planned inspection appropriate to risk	Regular monitoring over time	Check present structural performance	
	High	Significant corrosion likely, increasing with exposure period. Rate of corrosion could be high in parts*				
	Extremely high	Severe corrosion inevitable. Significant area likely to be affected				

Figure 6 Interpretation of steel reinforcement corrosion risk and prognosis

Notes: This interpretation applies to cases where corrosion is general in nature. Where severe pitting occurs the localised rate of corrosion can be significantly higher and may have implications for the structural capacity of the component concerned.

At a given chloride ion concentration, the risk of corrosion initiation may be higher for concretes containing pulverised fuel ash or silica fume.

* The chloride concentration and, hence, the risk of corrosion may increase with time.

** Degree of shading reflects level of action.



cracking parallel to the direction of the reinforcement throughout the length of the structural component. Fortunately, because the corrosion is fairly uniform, cracking of the concrete cover in normally reinforced or prestressed solid components usually occurs before the steel becomes excessively weak, so giving early visual warning of the deterioration.

Cracking in an atmospherically exposed concrete structure will usually be more severe when corrosion is caused by chlorides; sometimes rust staining will be evident at the cracks. Cracks will be wider than hairline and usually taper from the point of corrosion. The surface of the concrete may also bulge locally.

If chlorides are concentrated near the surface of the steel, or access of water and oxygen is restricted to a single location on the steel, severe pitting corrosion may occur. This reduces considerably the cross-sectional area of the bars at these locations, while the remainder of the bar may be left uncorroded. Structural cracking, or honeycombing, can also create conditions favourable to pitting corrosion by allowing the localised ingress of aggressive agents.

If a severely pitted bar is subjected to a tensile force, the strain is concentrated at what is effectively a notch and the overall strain on the bar will be less at failure than with a perfect bar. Hence, as notching becomes more severe, the reinforcement effectively becomes more brittle and large deformations which might normally give a warning of failure may not be seen.

Effect of corrosion on the performance of reinforced concrete elements

There are two general classes of failure. The first occurs when either the steel or the concrete is degraded by corrosion or cracking to a point where the materials can no longer support the stresses imposed on them. The second is where a disruption due to cracking alters significantly the geometry of the section (eg a longitudinal crack between the flange and web of an I-beam or hollow plank section).

In the first class of failure, it is usually serviceability rather than safety which is the major concern since visual signs of corrosion can be expected in normally reinforced and prestressed units before the reduction in the strength of the materials becomes critical. Cracking tends to occur earlier as the diameter of the reinforcing bar increases, for a given depth of cover and quality of concrete. As cracking proceeds the units will become less stiff which can lead to deflection in beams and bowing in slender columns.

The second class of failure is not so easy to

assess since less corrosion cracking is required to alter radically the component's behaviour. With this type of failure, cracking through the bottom of the web in hollow plank floor and roof units has been known to cause delamination of the soffits of the units. This significantly reduces the structural capacity of the units and also introduces the risk of falling debris.

Similarly, longitudinal cracking at the ends of prestressed beams may be important because in some designs no shear steel is provided, the shear resistance being provided by the plain concrete section and the prestressing force. In these circumstances, cracking at the ends of the units may reduce the shear capacity of the beams considerably and failure may occur without warning if the beam has been used as an isolated component.

The risk of spalling of the concrete in any circumstances due to reinforcement corrosion should be carefully assessed since the risk of falling lumps of concrete will have serious consequences for public safety.

Effect of corrosion on the performance of the overall structure

The deterioration of components incorporated into a structure and how it affects the overall performance must be considered as part of a much larger study of the stability of structures and their susceptibility to disproportionate or progressive collapse.

However, there are a few general points that can be made. Structural redundancy is widely accepted as beneficial for robustness and stability. Redundancy is usually achieved by introducing additional connections between components, either explicitly or by virtue of the construction type; for example, structural concrete floor toppings distribute loading into adjacent precast floor beams if the standard of workmanship is good. The load distribution and the strength benefits of redundancy can be obtained, provided most of the floor or surrounding structure is in good condition. This will often be the case where localised corrosion deterioration has taken place. It must also be recognised, though, that if general deterioration has occurred throughout a structure with a high degree of redundancy, connections between components could serve to spread partial collapse and damage, and not prevent it.

Corrosion deterioration may be found almost anywhere in a steel reinforced structure which, mainly due to loss of bond, can affect the bending and shear capacities of its components. It is therefore important to examine the overall structure for the consequences of both types of

failure mode and to concentrate upon the consequences of failure, rather than to consider the severity of the corrosion in isolation. Particularly sensitive are structures whose integrity and stability are wholly or partially dependent on high strength tendons. In these situations a small area of steel holds a large mass of construction in equilibrium. These types of structures include suspended shell roofs, long-span post-tensioned construction and tendon-stayed cantilevered structures.

Details which can lead to a brittle failure or act as a trigger for an extensive collapse will give particular concern. An example of this is the assessment of the punching shear resistance at the column-slab connection for flat slab structures.

Punching shear failure is characterised by the formation of radial cracking on the top surface followed by circumferential cracks at between 0.5 to 2 times the effective depth. In the absence of links, a key factor in providing resistance against punching shear is the tension layer of steel. In structures such as car parks, corrosion can lead to extensive delamination of the steel, and then to loss of bond strength and reduced punching shear resistance. When delamination is detected, a detailed structural assessment of the connection should be carried out prior to repair. If this evaluation indicates a strength below acceptable levels, a proper structural repair should be carried out with due regard for propping and other safety requirements.

References and further reading

- [1] **Concrete Bridge Development Group.** Testing and monitoring the durability of concrete structures. *Technical Guide No 2*. Crowthorne, CBDG, to be published in 2000.
- [2] **The Institution of Structural Engineers.** *Appraisal of existing structures* (2nd Edition). London, SETO Ltd, 1996.
- [3] **Transport Research Laboratory.** The half-cell potential method of locating corroding reinforcement in concrete structures. *Application Guide AG9*. Crowthorne, TRL.
- [4] **Browne R D.** *Durability of building materials. Vol 3, Design prediction of the life of reinforced concrete in marine and other chloride environments*. Amsterdam, Elsevier Science Publishing Co, 1982.
- [5] **Zimmermann L, Elsener B and Böhni H.** Critical factors for the initiation of rebar corrosion. *CD-ROM version of Proceedings of EUROCORR '99 Congress, Aachen, 30 August – 2 September 1999*.
- [6] **Vassie P.** Reinforcement corrosion and the durability of concrete bridges. *Proceedings of Institution of Civil Engineers, Part 1*, 76 (August 1984), 713–723.

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The structural adequacy and durability of large panel system dwellings. Part 1: Investigations of construction; Part 2: Guidance on appraisal. BR 107
Repair and maintenance of reinforced concrete. BR 254
Application of subsurface radar as an investigative technique. BR 340

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- 392 Assessment of existing high alumina cement concrete construction in the UK
- 405 Carbonation of concrete and its effect on durability
- 434 Corrosion of reinforcement in concrete: electrochemical monitoring

BRE Information Papers

- 11/98 Assessing carbonation depth in ageing high alumina cement concrete
- 21/86 Determination of the chloride and cement contents of hardened concrete

Highways Agency

- BD 63/94 Highway structures inspection and maintenance
- BA 63/94 Inspection of highway structures

American Society for Testing Materials

- ASTM C876-87. Standard test method for half cell potentials of reinforcing steel in concrete



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APPENDIX

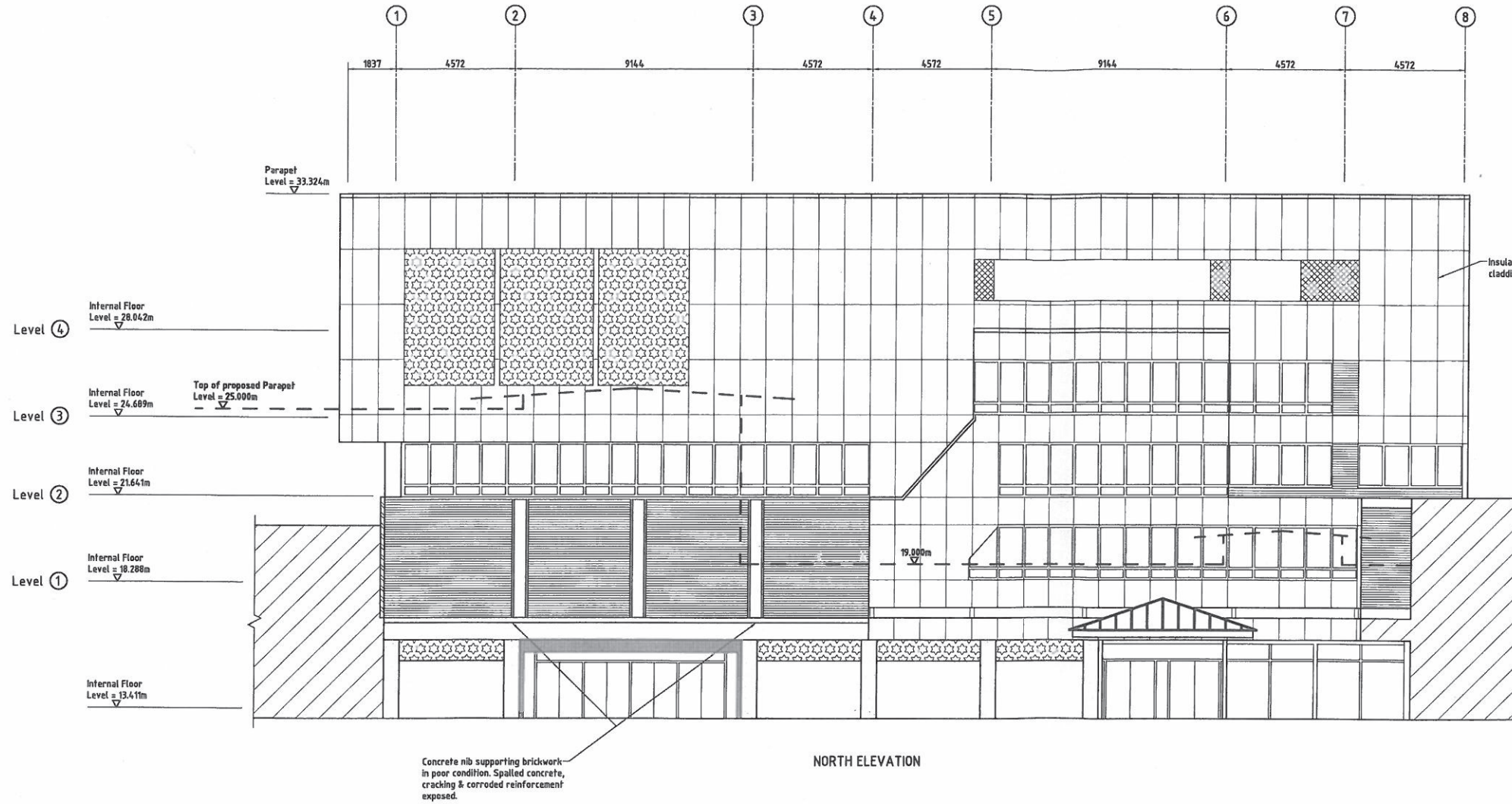
v) PREVIOUS ENGINEERS SURVEY DRAWINGS

NOTES

1. Defects shown on this drawing are based upon a binocular survey of the building carried out from ground level & adjacent buildings, September 2001.
2. Steel window frames require repainting. General condition of paintwork poor, peeling in places with signs of rusting to frames in some places.
3. External sills to windows within brick panels may be asbestos cement. Signs of fibrous materials in sills.
4. Levels are based on Record Drawings.

LEGEND

- SP Spall
C Crack
Area of precast unit apparently made good (probably during construction).
R Small patch repair
Heavy mortar loss to bed joint



Ref.	Amendment	Ch'd	Date
Stirling Maynard & Partners, Consulting Engineers, Stirling House, Rightwell, Bretton, Peterborough, PE3 8DJ Telephone: 01733 262319 Fax: 01733 331027 Email: enquiries@stirlingmaynard.com			
Client:			
NEWPORT CITY COUNCIL			
Scheme Title:			
CENTRAL LIBRARY, MUSEUM AND ART GALLERY - EXTERNAL FABRIC			
Drawing Title:			
NORTH ELEVATION - 2007 DEFECT DRAWING			
Project Manager	PETER BEVERIDGE	Drawing Status	FINAL SURVEY
Drawn	D.Farley-Scott	Traced	N/A
Checked	P.G.B.	Scale	1/75 @ A0
Date	28/09/07		
			4284/02/01

NOTES

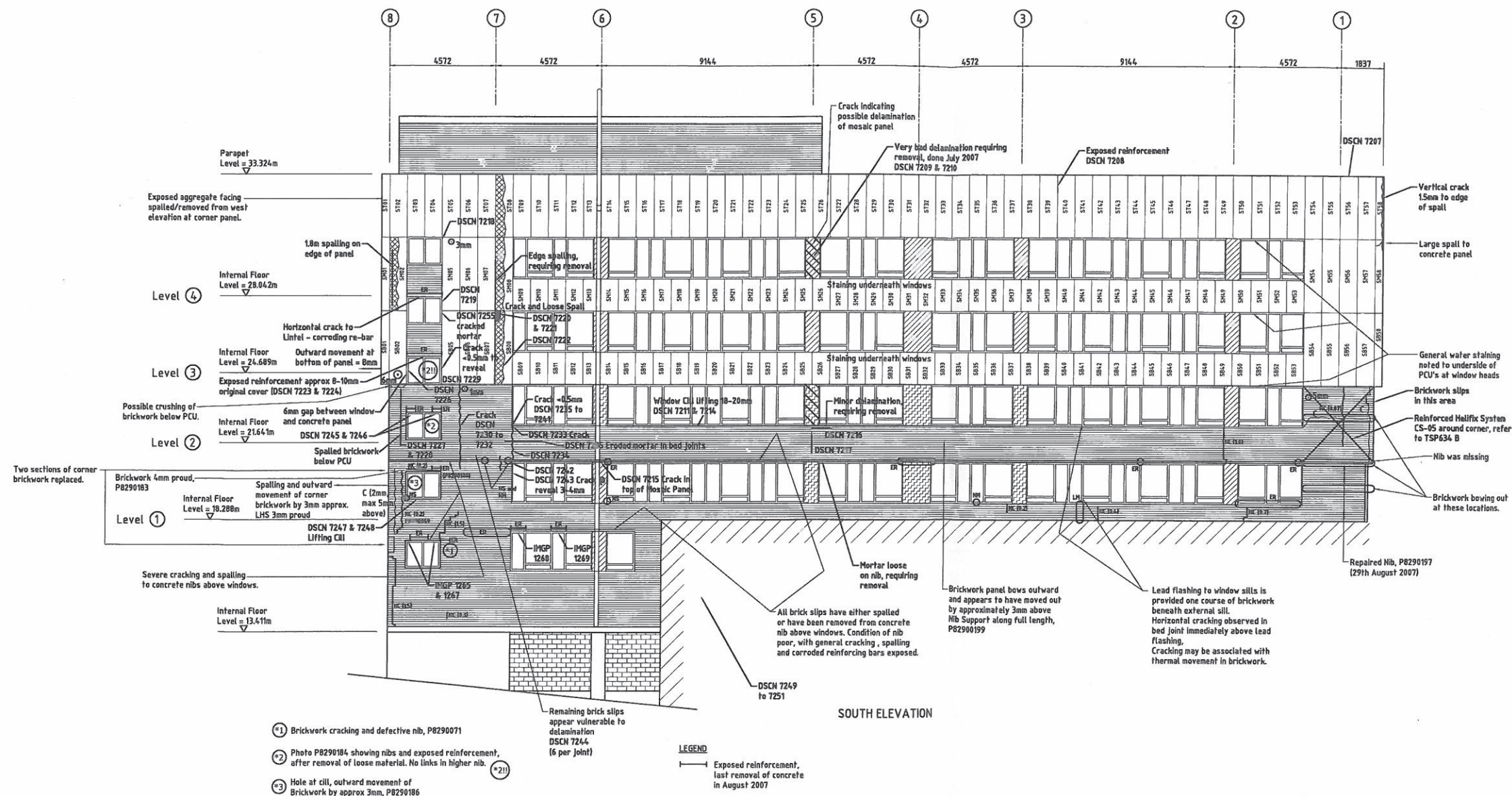
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2. Steel window frames require repainting. General condition of paintwork poor, peeling in places with signs of rusting to frames in some places.
3. External sills to windows within brick panels may be asbestos cement. Signs of fibrous materials in sills.
4. Levels are based on Record Drawings.

LEGEND - (based on 13th to 15th June 2007 Defect Survey)

- LM Loose mortar
NM No Mortar
RS Reinforcement Staining
ER Exposed Reinforcement
NS New Spalling to brickwork
PR Poor Repair
NC (0.2) New Cracking (with crack widths in "mm")

LEGEND - (based on September 2001 Defect Survey)

- SP Spall
C Crack
Area of precast unit apparently made good (probably during construction).
R Small patch repair
Heavily mortar loss to bed joint



NOTES

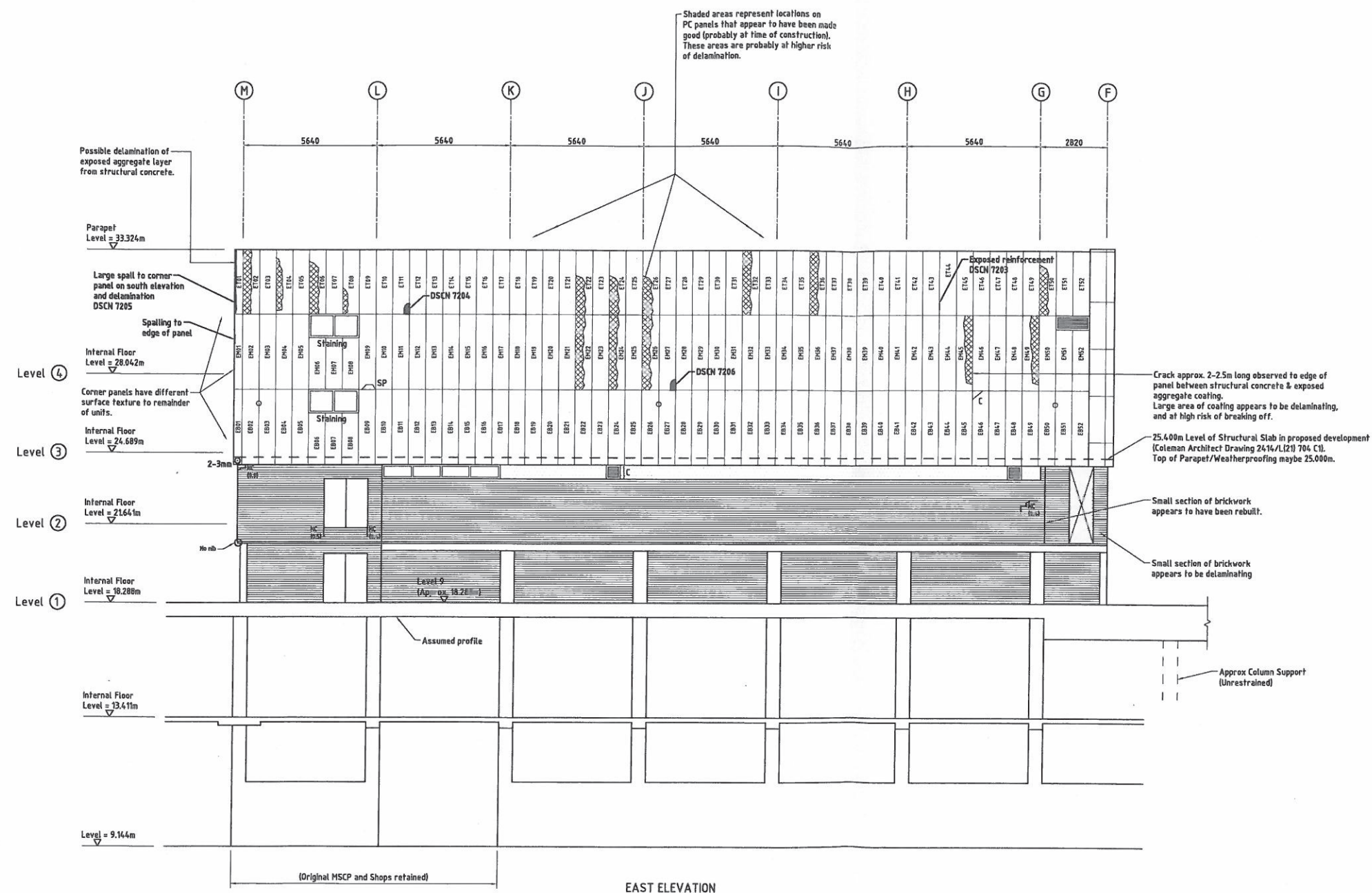
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2. Steel window frames require repainting. General condition of paintwork poor, peeling in places with signs of rusting to frames in some places.
3. External sills to windows within brick panels may be asbestos cement. Signs of fibrous materials in sills.
4. Levels are based on Record Drawings.

LEGEND - (based on 13th to 15th June 2007 Defect Survey)

- LM Loose mortar
NM No Mortar
RS Reinforcement Staining
ER Exposed Reinforcement
NS New Spalling to brickwork
PR Poor Repair
NC (0.2) New Cracking (with crack widths in "mm")

LEGEND - (based on September 2001 Defect Survey)

- SP Spall
C Crack
Area of precast unit apparently made good (probably during construction).
R Small patch repair
Heavy mortar loss to bed joint



EAST ELEVATION

Ref.	Amendment	Ch'd	Date
Stirling Maynard & Partners, Consulting Engineers, Stirling House, Rightwell, Bretton, Peterborough, PE3 8DJ Telephone: 01733 282319 Fax: 01733 331527 Email: enquiries@stirlingmaynard.com			
Client			
NEWPORT CITY COUNCIL			
Scheme Title			
CENTRAL LIBRARY, MUSEUM AND ART GALLERY - EXTERNAL FABRIC			
Drawing Title			
EAST ELEVATION - 2007 DEFECT DRAWING			
Project Manager		Drawing Status	
PETER BEVERIDGE		FINAL SURVEY	
Drawn DJFarley-Sell		Drawing No.	
Checked P.G.B.		4284/02/03	
Date 28/09/07		Scale 1/75 @ A0	

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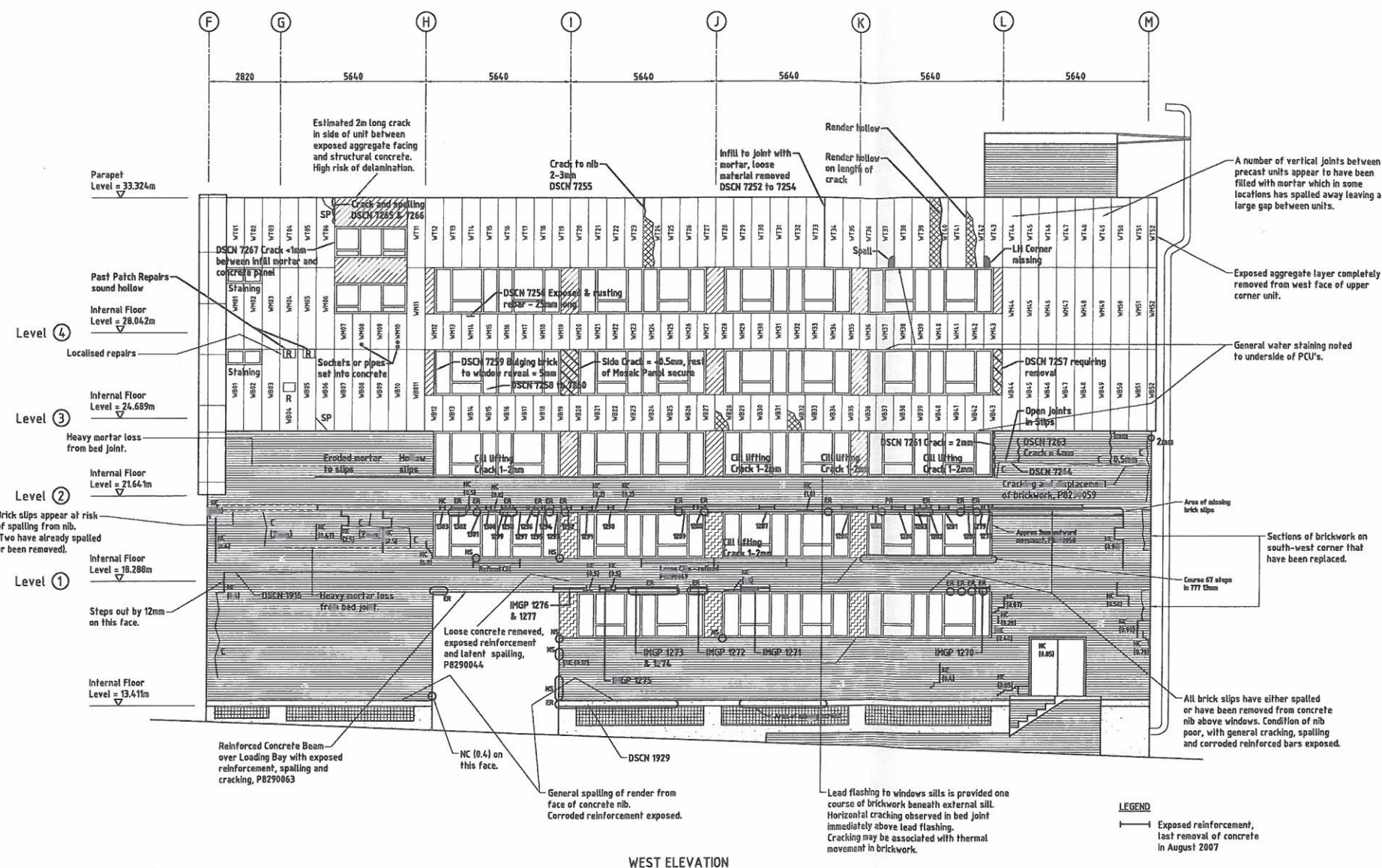
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2. Steel window frames require repainting. General condition of paintwork poor, peeling in places with signs of rusting to frames in some places.
3. External sills to windows within brick panels may be asbestos cement. Signs of fibrous materials in sills.
4. Levels are based on Record Drawings.

LEGEND - (based on 13th to 15th June 2007 Defect Survey)

- LM Loose mortar
NM No Mortar
RS Reinforcement Staining
ER Exposed Reinforcement
NS New Spalling to brickwork
PR Poor Repair
NC (0.2) New Cracking (with crack widths in "mm")

LEGEND - (based on September 2001 Defect Survey)

- SP Spall
C Crack
Area of precast unit apparently made good (probably during construction).
R Small patch repair
Heavy mortar loss to bed joint



WEST ELEVATION

Ref.	Amendment	Ch'd	Date
Stirling Maynard & Partners, Consulting Engineers, Stirling House, Rightwell, Bretton, Peterborough, PE3 8DJ Telephone: 01733 282319 Fax: 01733 331827 Email: enquiry@stirlingmaynard.com			
Client			
NEWPORT CITY COUNCIL			
Scheme Title			
CENTRAL LIBRARY, MUSEUM AND ART GALLERY - EXTERNAL FABRIC			
Drawing Title			
WEST ELEVATION - 2007 DEFECT DRAWING			
Project Manager	PETER BEVERIDGE	Drawing Status	FINAL SURVEY
Drawn	DJFarley-Smith	Traced	N/A
Checked	P.G.B.	Scale	1/75 @ A0
Date	28/09/07		4284/02/04

APPENDIX

vi) CONCRETE REPAIR PRODUCTS

High performance fibre reinforced medium weight concrete reinstatement mortar conforming to the requirements of BS EN 1504-3 Class R4

Uses

For the reinstatement of reinforced concrete where low permeability characteristics are required and where higher compressive strength is a consideration.

Renderoc HBS has been specifically developed for the repair of columns and beams but, because of its relatively low fresh wet density, is also suitable for soffit repairs.

Where compatibility with lower strength concrete is required, Renderoc HB30 should be used.

Renderoc HBS is suitable for repair methods 3.1, 3.3, 4.4, 7.1, 7.2 as defined by BS EN 1504-3.

Advantages

- Compatible with concrete of compressive strength 35 - 60 N/mm²
- High-build applications possible while maintaining higher compressive strengths — fewer cold joints
- Frequently obviates the need for formwork
- Polymer-modification provides extremely low permeability to water, carbon dioxide and chlorides
- Shrinkage compensated to provide long-term dimensional stability
- Can be applied quickly and efficiently by wet spraying
- One component, pre-bagged to overcome site-batched variations
- Contains no chloride admixtures

Description

Renderoc HBS is supplied as a ready to use blend of dry powders which requires only the site addition of clean water to produce a highly consistent, medium-weight repair mortar.

It is based on Portland cements, graded aggregates, lightweight fillers and chemical additives which provide a mortar with good handling characteristics while minimising water demand. The low water requirement ensures good strength gain and long-term durability.

Renderoc HBS has been specifically engineered for vertical and overhead repair work. It can be applied in sections up to 40 mm thickness in vertical locations and up to 30 mm thickness in overhead locations in a single application without the use of formwork.

Thicker sections can be achieved by the use of formwork or can be built up in layers. Deep pockets can sometimes be filled in a single application dependent on the configuration of the pocket and the volume of exposed reinforcing steel.

Renderoc HBS can be quickly and efficiently applied by the wet spray technique. Consult the local Fosroc office for further information.

The material should not be applied at less than 10 mm thickness.

Standards compliance

Renderoc HBS complies with the classification R4 according to EN 1504-3, repair methods 3.1, 3.3, 4.4, 7.1 and 7.2.

 0370	
Fosroc Ltd Drayton Manor Business Park, Coleshill Road, Tamworth, B78 3XN, UK 09 DOP: UK9-51 0370-CPR-0845	
Fosroc® Renderoc HBS	
EN 1504:3 Structural and non-structural repair methods 3, 4 and 7	
Compressive strength	Class R4: ≥ 45 MPa
Chloride ion content	$\leq 0.05\%$
Adhesion strength by pull-off test	≥ 2.0 MPa
Thermal compatibility: freeze thaw cycling with immersion	≥ 2.0 MPa
Carbonation resistance	Pass
Elastic modulus	≥ 20 GPa
Reaction to fire	Class A2
Dangerous substances	Complies with 5.4

Fosroc® Renderoc HBS

Properties

The following results were obtained at a water : powder ratio of 0.13 and a temperature of 20°C unless otherwise stated.

Test method	Standard	EN 1504 R4 Requirement	Result
Compressive Strength	EN 12190:1999	≥ 45 MPa	@ 1 Day 20 MPa @ 28 days 50 MPa
Bond strength by pull off:	EN 1542:1999	≥ 2.0 MPa	2.1 MPa
Chloride ion content:	EN 1015-17:2000	≤ 0.05 %	0.03%
Freeze thaw cycling:	EN 13687-1:2002	≥ 2.0 MPa	2.0 MPa
Resistance to carbonation d_k	EN 13295:2005	$\leq$ ref concrete	Complies
Elastic Modulus in Compression	EN 13412	≥ 20 GPa	25 GPa @ 28 Days
Fire rating	EN 13501-1		Class A2 s1 d0 Non-Combustible
Flexural strength	BS 6319 Pt 3:1990	-	6.5 MPa @ 28 days
Tensile strength	BS 6319 Pt 3:1985	-	2.9 MPa @ 28 days
Setting time	BS 4551 Pt 14:1980	-	Initial set: 3 hours Final set: 5 hours
Fresh wet density		-	Nominally 2000 kg/m ³
Build Characteristics hand applied Minimum thickness: Overhead: Vertical:	- - -	- - -	10 mm Up to 30 mm Up to 40 mm
Chemical resistance		-	The low permeability of Renderoc HBS severely retards chemical attack in aggressive environments. The cured mortar is impermeable to acid gases, waterborne chloride ions and oxygen.
Chloride ion ingress	EN 13396	-	0.146% after 6 months in 3% NaCl solution at 8-10mm depth

Clarification of property values: The typical properties given above are derived from laboratory testing. Results derived from testing field applied samples may vary.

Fosroc® Renderoc HBS

Application instructions

Preparation

* Saw cut the edges of the repair to a depth of at least 10 mm to provide a square edge. Break out the complete repair area to a minimum depth of 10 mm up to the sawn edge.

Clean the surface and remove any dust, unsound or contaminated material, plaster, oil, paint, grease, corrosion deposits or algae. Where breaking out is not required, roughen the surface and remove any laitance by light scabbling or abrasive-blasting.

Oil and grease deposits should be removed by steam cleaning, detergent scrubbing or the use of a proprietary degreaser. The effectiveness of decontamination should then be assessed by a pull-off test.

Expose fully any corroded steel in the repair area and remove all loose scale and corrosion deposits. Steel should be cleaned to a bright condition paying particular attention to the back of exposed steel bars. Abrasive-blasting is recommended for this process.

Where corrosion has occurred due to the presence of chlorides, the steel should be high-pressure washed with clean water immediately after abrasive-blasting to remove corrosion products from pits and imperfections within its surface.

*(Refer to HSE information sheet CIS36 regarding control of exposure to construction dust, available at www.hse.gov.uk)

Reinforcing steel priming

Apply one full coat of Nitoprime Zincrich Plus and allow to dry before continuing. If any doubt exists about having achieved an unbroken coating, a second application should be made and, again, allowed to dry before continuing.

Concrete Priming

The concrete substrate should be saturated surface dry immediately before the application of the primer i.e. it should be thoroughly saturated with clean water and any residual surface water removed prior to applying Renderoc Primer.

Under severe drying conditions repeated soaking may be necessary to ensure the substrate is still saturated at the time of application of the primer.

Add 3 parts by volume of Renderoc Primer Part A to 1 part by volume Renderoc Primer Part B (4 to 1 by weight) in a clean mixing vessel and mix with a spatula or slow speed drill until a homogeneous smooth paste is produced.

Scrub Renderoc Primer slurry into the surface.

Renderoc HBS can be applied immediately to the primed surface. If the primer dries apply another coat of primer before continuing.

Open bags of Renderoc Primer Part A should be sealed and stored in a dry area. Use within 14 days.

In exceptional circumstances, e.g. where a substrate/repair barrier is required or where the substrate is water immersed or likely to remain permanently damp, Nitobond EP bonding aid should be used. Contact the local Fosroc office for further information.

Mixing

Care should be taken to ensure that Renderoc HBS is thoroughly mixed. A forced-action mixer is essential. Mixing in a suitably sized drum using an approved Renderoc Mixing Paddle (MR4) in a slow speed (400/500 rpm) heavy-duty drill is acceptable for the occasional one-bag mix.

Free-fall mixers must not be used. Mixing of part bags should never be attempted.

For normal applications, place 3.25 to 3.5 litres of drinking quality water into the mixer.

With the machine in operation, add one full bag of Renderoc HBS and mix, for a minimum of 3 minutes and a maximum of 5 minutes, until fully homogeneous.

The consistency may be adjusted by the addition of small amounts of water up to the maximum total water content of 3.5litres.

Note that the powder must always be added to the water.

Mixing warning

As with other 'one pack' repair mortars, Renderoc HBS may exhibit satisfactory handling characteristics even though inadequately mixed. This will result in a significantly lower level of performance or possible failure. It is therefore essential that mixing instructions are strictly adhered to with particular emphasis on the quantity of water used and the time of the mixing operation.

Application

Exposed steel reinforcing bars should be firmly secured to prevent movement during application.

Apply the mixed Renderoc HBS by gloved hand or trowel, thoroughly compacting onto the primed substrate and around exposed reinforcement.

If sagging or slumping occurs the Renderoc HBS should be completely removed and reapplied at a reduced thickness to a correctly primed substrate.

Build-up

Additional build-up can be achieved by application of multiple layers.

The surface of the intermediate layers should be comb scratch-keyed and cured with Nitobond AR. Repriming with Renderoc Primer and a further application of Renderoc HBS may proceed as soon as this layer has set.

Spray application

Renderoc HBS can be quickly and efficiently applied by the wet spray technique. In circumstances where large areas

Fosroc® Renderoc HBS

of repair are required, the rapid placement and higher build attainable by this method offer economic advantages over hand-trowelling. The resultant repair also offers a generally more dense compound with enhanced mortar/substrate bond characteristics.

For further details on wet spray techniques contact the local Fosroc office.

Finishing

Renderoc HBS is finished by striking off with a straight edge and closing with a steel float. Wooden or plastic floats, or damp sponges, may be used to achieve the desired surface texture. The completed surface should not be overworked. After spray application, the mortar may need to be 'cut back' to the required profile using a steel float and then finished with damp sponges as described above.

Low temperature working

In cold conditions down to 5°C, the use of warm mixing water (up to 30°C) is advisable to accelerate strength development. Normal precautions for winter working with cementitious materials should then be adopted. The material should not be applied when the substrate and/or air temperature is 5°C and falling. At 5°C static temperature or at 5°C and rising, the application may proceed.

High temperature working

At ambient temperatures above 35°C, the material should be stored in the shade and cool water used for mixing.

Curing

Renderoc HBS is a cement-based repair mortar. In common with all cementitious materials, it must be cured immediately after finishing in accordance with good concrete practice. The use of Nitobond AR, sprayed on to the surface of the finished mortar in a continuous film, is recommended. A low pressure atomising sprayer is essential for applying the Nitobond AR. Any excessive run-off on verticals or drips on soffits should be removed by brush before they harden.

Large areas should be cured as trowelling progresses (0.5m² at a time) without waiting for completion of the entire area.

In fast drying conditions, supplementary curing with polythene sheeting taped down at the edges must be used. In cold conditions, the finished repair must be protected from freezing.

Overcoating with protective decorative finishes

Renderoc HBS is extremely durable and will provide long-term protection to the embedded steel reinforcement within the repaired locations. The surrounding parts of the structure will benefit from the application of a barrier/decorative coating to limit the advance of chlorides and carbon dioxide, bringing them to the same protective standard as the repair itself. Fosroc recommend the use of the Dekguard range

of protective, anti-carbonation coatings. These products provide a decorative and uniform appearance as well as protecting areas of the structure which might otherwise be at risk from the environment. Dekguard products may be applied over the repair area without prior removal of the Nitobond AR curing membrane. Other curing membranes must be removed prior to the application of Dekguard products.

Cleaning

Renderoc HBS and Renderoc Primer should be removed from tools, equipment and mixers with clean water immediately after use. Cured material can only be removed mechanically.

Clean tools used with Nitoprime Zincrich Plus and Nitobond EP before material cures, using Fosroc Solvent 102.

Estimating

Supply

Renderoc HBS:	25 kg bags
Nitoprime Zincrich Plus:	1.9 litre and 800 ml cans
Renderoc Primer: Handy pack :	20 Kg Part A + 5 litres part B 2.5 litres - 2m ² / litre
Nitobond AR:	5 and 25 litre drums
Fosroc Solvent 102:	5 and 25 litre tins

Coverage and yield

Renderoc HBS:	Approximately 15 litres / 25kg bag (approximately 1.5 m ² at 10 mm thickness)
Nitoprime Zincrich Plus:	8 m ² /litre
Nitobond AR:	6 - 8 m ² / litre
Renderoc Primer: Handy pack:	Approximately 13 litres-2m ² /litre 2.5 litres - 2m ² / litre

Notes: the actual yield per bag of Renderoc HBS will depend on the consistency used. The yield will be reduced if the material is applied by a spray technique.

Limitations

Renderoc HBS should not be used when the temperature is below 5°C and falling. Do not mix part bags. Due to the lightweight nature of Renderoc HBS, the product should not be used in areas subjected to traffic (in these circumstances, Renderoc S should be considered).

Renderoc HBS should not be exposed to moving water during application. Exposure to heavy rainfall prior to the final set may result in surface scour.

If any doubts arise concerning temperature or substrate conditions, consult the local Fosroc office.

Fosroc® Renderoc HBS

Storage

The product has a shelf life of 12 months from the date of manufacture if kept in dry storage in the original, unopened bags. If stored at high temperatures and/or high humidity the shelf life may be reduced to less than 6 months.

Nitobond AR and Renderoc Primer Part B should be protected from frost.

Precautions

Health and safety

For further information refer to the appropriate Safety Data Sheets available at www.fosroc.com.

Fire

Renderoc HBS is non-flammable.

Nitoprime Zincrich Plus and Fosroc Solvent 102: are flammable. Keep away from sources of ignition. No Smoking. In the event of fire, extinguish with CO₂ or foam. Do not use a water jet

Flash points

Nitoprime Zincrich Plus:	41°C
Fosroc Solvent 102:	33°C

Fosroc and Renderoc are trademarks of
 Fosroc International Limited



Important note

Fosroc products are guaranteed against defective materials and manufacture and are sold subject to its standard Conditions for the Supply of Goods and Services, copies of which may be obtained on request. Whilst Fosroc endeavours to ensure that any advice, recommendation, specification of information it may give is accurate and correct, it cannot, because it has no direct or continuous control over where or how its products are applied, accept any liability either directly or indirectly arising from the use of its products, whether or not in accordance with any advice, specification, recommendation of information given by it.

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Certificate number FM 610

Fosroc® Nitomortar S

constructive solutions

High strength, abrasion-resistant epoxy reinstatement mortar

Uses

For the fast and permanent reinstatement of concrete, particularly where high strength, abrasion-resistance and resistance to chemicals is required. The product is designed for horizontal use but can be applied vertically, although generally in thinner sections. It is ideally suited for acid tanks, sea walls, industrial floors and for use as a bedding mortar. Nitomortar S can be used for emergency repairs where fast strength gain is important. When properly compacted, the mortar is highly impermeable.

In certain instances, Nitomortar S can be used on metal substrates. Contact the local Fosroc office for advice in this respect.

For fast repairs in vertical and overhead locations where a higher-build, lightweight formulation is required, the use of Nitomortar HB is recommended.

Advantages

- High ultimate strength — suitable for structural use
- Early development of strength minimises disruption
- Abrasion-resistance — suitable for heavily trafficked areas
- Highly resistant to a wide range of chemicals
- Cured product is highly impermeable to water when 'tight trowel' finished
- Typically twice as strong as good quality concrete
- Pre-weighed components ensure consistency

Description

Nitomortar S is based on a high quality solvent-free epoxy resin system. The special silica aggregates provide high strength and excellent abrasion resistance. Nitomortar S is a three-component material supplied in pre-weighed quantities ready for on-site mixing and use.

Properties

The following results were obtained at a temperature of 20°C unless otherwise specified.

Test method	Typical result
Compressive strength (BS 6319, Pt 2):	70 N/mm ² @ 7 days
Flexural strength (BS 6319, Pt 3):	20 N/mm ² @ 7 days
Tensile strength (ASTM C 307):	10 N/mm ² @ 7 days
Compressive modulus (ASTM C 469):	16.2 kN/mm ² @ 28 days
Water absorption:	0.2% (concrete - 5.0%)
Pot life:	45 minutes @ 20°C 20 minutes @ 35°C
Initial hardness:	24 hours
Full cure:	7 days
Fresh wet density:	Approximately 2015 kg/m ³ (fully compacted)
Chemical resistance:	The low permeability of Nitomortar S retards chemical attack in aggressive environments

Performance of Nitomortar S blocks continually immersed at 20°C:

Citric acid	10%	Excellent
Tartaric acid	10%	Excellent
Hydrochloric acid	25%	Excellent
Sodium hydroxide	50%	Excellent
Diesel fuel / petrol	100%	Excellent
Sulphuric acid	10%	Very good
Sugar solutions	Saturated	Very good
Lactic acid	10%	Very good
Hydrocarbons	100%	Very good
Phosphoric acid	50%	Very good
Nitric acid	10%	Good
Acetic acid	5%	Limited

Fosroc® Nitomortar S

Application instructions

Preparation

Clean the surface and remove any dust, unsound material, plaster, oil, paint, grease, corrosion deposits or algae. Roughen the surface and remove any laitance by light scabbling or grit-blasting. Saw cut or cut back the extremities of the repair locations to a depth of at least 5 mm to avoid feather-edging and to provide a square edge. Break out the complete repair area to a minimum depth of 5 mm up to the sawn edge.

Oil and grease deposits should be removed by steam cleaning, detergent scrubbing or the use of a proprietary degreaser. The effectiveness of decontamination should then be assessed by a pull-off test.

Expose fully any corroded steel in the repair area and remove all loose scale and corrosion deposits. Steel should be cleaned to a bright condition paying particular attention to the back of exposed steel bars. Grit-blasting is recommended for this process.

Reinforcing steel priming

The cleaned steel should be coated within 3 hours. Apply one full coat of Nitoprime Zincrich Plus and allow to dry before continuing. If any doubt exists about having achieved an unbroken coating, a second application should be made and, again, allowed to dry before continuing.

Substrate priming

Horizontal or damp substrates should be primed using Nitoprime 25. Predominantly vertical and metal substrates should be primed using Nitoprime 28. In both cases, the primer should be mixed in the proportions supplied, adding the entire contents of the 'hardener' tin to the 'base' tin. The two components should be thoroughly mixed together for 3 minutes.

The mixed primer should be scrubbed well into the prepared substrate, taking care that all imperfections in the surface are properly coated and avoiding 'puddling' in depressions. If the primer is absorbed within 30 minutes, a second coat should be applied before continuing. Nitomortar S can be applied as soon as the primer has started to gel but still has surface 'tack'. This is normally between 30 minutes and 4 hours dependent on the ambient and substrate temperatures. If the primer cures hard, a second application must be made before application of Nitomortar S. The usable life of the mixed primer is approximately 60 minutes at 20°C or 30 minutes at 35°C.

Mixing

Care should be taken to ensure that Nitomortar S is thoroughly mixed to produce a fully homogeneous, trowellable mortar.

'Standard' packs may be mixed by hand. The 'hardener' and 'base' components should be stirred thoroughly in order to

disperse any settlement before mixing them together. The entire contents of the 'hardener' and 'base' containers should then be emptied into the plastic bucket and thoroughly mixed for 3 minutes. While mixing continuously, the entire bag of aggregate should then be added slowly and the three components blended together for a further 2 to 3 minutes, ensuring that the aggregate is thoroughly wetted out with the mixed resins. Under no circumstances should part packs be used.

'Industrial' packs must be mixed mechanically. Again, the 'hardener' and 'base' components should be stirred thoroughly in order to disperse any settlement before mixing them together. The entire contents of the 'hardener' container should then be emptied into the 'base' container and thoroughly mixed for 3 minutes, then emptied into a forced action mixer of adequate capacity (e.g. Daines, Cretangle or Pennine). Add the aggregate slowly with the mixer running and continue for 2 to 3 minutes until all the components are thoroughly blended.

Under no circumstances should part packs be used.

Application

Apply the mixed Nitomortar S to the prepared substrate by wood float, pressing firmly into place to ensure positive adhesion and full compaction. Thoroughly compact the mortar around any exposed reinforcement. In restricted locations, or where exposed reinforcing steel is present, application by gloved hands is an acceptable alternative but, in all cases, the product must be finished to a tight surface with a steel trowel. Nitomortar S can be applied in sections up to 50 mm thickness in horizontal locations or up to 6 mm thickness in vertical locations in a single application and without the use of formwork. Thicker vertical sections may sometimes be possible dependent on the profile of the substrate and the volume of exposed reinforcing steel but should generally be built up in layers.

When larger areas are being rendered, a chequerboard application technique is recommended.

Note: the minimum applied thickness of Nitomortar S is 5 mm.

Build-up

Additional build-up can be achieved by application of multiple layers. Exposed steel reinforcing bars should be firmly secured to avoid movement during the application process as this will affect mortar compaction, build and bond.

Where thicker sections are required, the surface of the intermediate applications should be scratch-keyed to provide a suitable surface for subsequent layers. The application of additional layers should follow between 8 and 24 hours (@ 20°C) after the first application. This time should be reduced at higher temperatures. Repriming and a further application of Nitomortar S may then proceed.

If sagging occurs during application, the Nitomortar S should be completely removed and reapplied at a reduced thickness on to the correctly reprimed substrate.

Fosroc® Nitomortar S

Finishing

Nitomortar S is finished by the use of a wood float and closed with a steel trowel. The completed surface should not be overworked.

Low temperature working

Nitoprime 25, Nitoprime 28 and Nitomortar S can be applied in cold conditions down to 5°C. The materials should not be applied when the substrate and/or air temperature is 5°C and falling. At 5°C static temperature or at 5°C and rising, the application may proceed.

High temperature working

At ambient temperatures above 35°C, Nitoprime 25, Nitoprime 28 and Nitomortar S will have shorter pot lives and working lives. The materials should be stored in the shade or in an air-conditioned environment and should not be applied in direct sunlight.

Curing

Unlike cementitious materials Nitomortar S does not require curing immediately after finishing, but does require protection from rain and wet conditions during the initial 24 hours after placement.

Overcoating with protective/decorative finishes

Nitomortar S is extremely durable and resistant to a wide range of acids, alkalis and industrial chemicals and will provide excellent protection to the concrete and embedded steel reinforcement within the repaired locations. The surrounding parts of the structure may benefit from the application of a protective coating, thus bringing them up to the same protective standard as the repair itself. Fosroc recommend the use of the Nitocote range of epoxy resin, chemical-resistant, protective coatings.

For surrounding areas not subjected to chemical attack or physical wear, Fosroc recommend the use of the Dekguard range of anti-carbonation, anti-chloride protective coatings. These products provide a decorative and uniform appearance as well as protecting areas of the structure which might otherwise be at risk from the environment.

Nitocote epoxy resin protective coatings should be applied within 24 hours. Dekguard products should not be applied until the Nitomortar S is at least 3 days old. For further advice, consult the local Fosroc office.

Cleaning

Nitoprime Zincrich Plus, Nitoprime 25, Nitoprime 28 and Nitomortar S should be removed from tools, equipment and mixers with Fosroc Solvent 102 immediately after use.

Estimating

Supply

Nitomortar S:	5 kg 'Standard' packs 16 kg 'Industrial' packs
Nitoprime Zincrich Plus:	1.9 litre and 800ml cans
Nitoprime 25:	0.95 kg 'Handy' packs 5 kg 'Industrial' packs
Nitoprime 28:	0.45 kg 'Handy' packs 4.2 kg 'Industrial' packs
Fosroc Solvent 102:	5 and 25 litre tins

Coverage and yield

Nitomortar S:	2.5 litres / 5 kg 'Standard' pack 8.0 litres / 16 kg 'Industrial' pack
Nitoprime Zincrich Plus:	8 m ² /litre
Nitoprime 25:	5 m ² / 0.95 kg 'Handy' pack 26 m ² / 5 kg 'Industrial' pack
Nitoprime 28:	2.4 m ² / 0.45 kg 'Handy' pack 20 m ² / 4.2 kg 'Industrial' pack

Notes: the coverage figures for Nitoprime Zincrich Plus and Nitoprime 28 are theoretical — due to wastage factors and the variety and nature of possible substrates, practical coverage figures will be reduced.

Limitations

Nitomortar S should not be used when the temperature is below 5°C and falling. Do not mix part packs under any circumstances. Nitomortar S should not be used in overhead locations — use Nitomortar HB for this purpose. Nitomortar S should not be exposed to moving water during application. Exposure to heavy rainfall prior to the final set will result in surface softening and scour. Nitoprime 28 is not a damp tolerant primer. If any doubts arise concerning temperature or substrate conditions, consult the local Fosroc office.

Storage

Store in dry conditions in the original, unopened bags or packs. All products have a shelf life of 18 months at 20°C if kept in a dry store in the original, unopened bags or packs.

If stored at high temperatures, the shelf life may be reduced to 4 to 6 months.

Fosroc® Nitomortar S

Precautions

Health and safety

For further information refer to the appropriate Product Safety Data Sheet.

Fire

Nitomortar S is non-flammable.

Nitoprime Zincrich Plus, Nitoprime 28 and Fosroc Solvent 102 are flammable. Keep away from sources of ignition. No smoking. In the event of fire, extinguish with CO₂ or foam. Do not use a water jet.

Flash points

Nitoprime Zincrich Plus:	41°C
Fosroc Solvent 102:	33°C
Nitoprime 28	27°C

For further information, refer to the Product Safety Data Sheets.

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Important note

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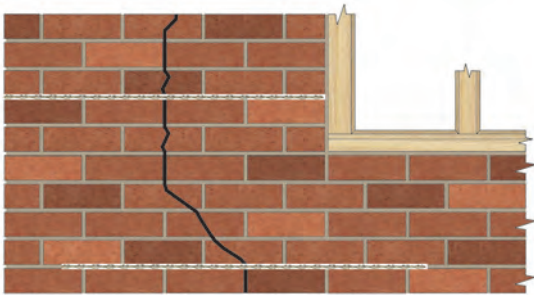


Crack Stitching

A reliable and cost-effective means of repairing and stabilising cracked masonry

Applications

- Rapid and permanent solution to cracked masonry
- Suitable for all forms of masonry structure



Over 100 standard repair specifications are available online, covering all common structural faults.
Relevant Repair Details: RDs CS01 to CS14



For full Product Information, Case Studies and downloadable Repair Details go to:

www.helifix.co.uk/products/remedial-products/crack-stitching/



Features

- Fully concealed, non-disruptive repair solution
- More reliable than crack injection methods
- HeliBond cementitious grout is injectable and rapidly produces high compressive strength
- HeliBars and HeliBond grout combine to create excellent tensile strength within the masonry
- No additional stresses are introduced during installation
- Masonry remains flexible enough to accommodate normal building movement
- Tensile loads are redistributed
- Reduces likelihood of further cracking nearby
- Avoids costly and disruptive taking down and rebuilding



HeliBar is inserted into HeliBond grout within a cut slot

Installation Procedures

1. HeliBar to be long enough to extend a minimum of 500mm either side of the crack or 500mm beyond the outer cracks if two or more adjacent cracks are being stitched using one rod.
2. Where a crack is less than 500mm from the end of a wall or an opening, the HeliBar is to be continued for at least 200mm around the corner and bonded into the adjoining wall or bent back and fixed into the reveal, avoiding any DPC.
3. For solid masonry in excess of 300mm thick and in a cavity wall where both leaves are cracked, the wall must be crack stitched on both sides.
4. If there is render, this thickness must be added to the depth of slot. Crack stitching must be installed in the masonry and never in the render.
5. Ensure the masonry is well wetted or primed to prevent premature drying of the HeliBond due to rapid de-watering, especially in hot conditions. Ideally additional wetting of the slot should be carried out 1 to 2 minutes prior to injecting the HeliBond grout.
6. Do not use HeliBond when the air temperature is +4°C and falling or apply over ice. In all instances the slot must be thoroughly damp or primed prior to injection of the HeliBond grout.



1. Rake out or cut slots into the horizontal mortar beds, a minimum of 500mm either side of the crack



4. Using the HeliBar Insertion Tool, push one HeliBar into the grout to obtain good coverage



2. Clean out slots and flush with clean water and thoroughly soak the substrate within the slot



5. Insert a further bead of HeliBond over the exposed HeliBar, finishing 12mm from the face, and 'iron' firmly into the slot using the HeliBar Insertion Tool



3. Using the Helifix Pointing Gun, inject a bead of HeliBond along the back of the slot



6. Re-point the mortar bed and make good the vertical crack with CrackBond TE3

Slot Depth and Spacing

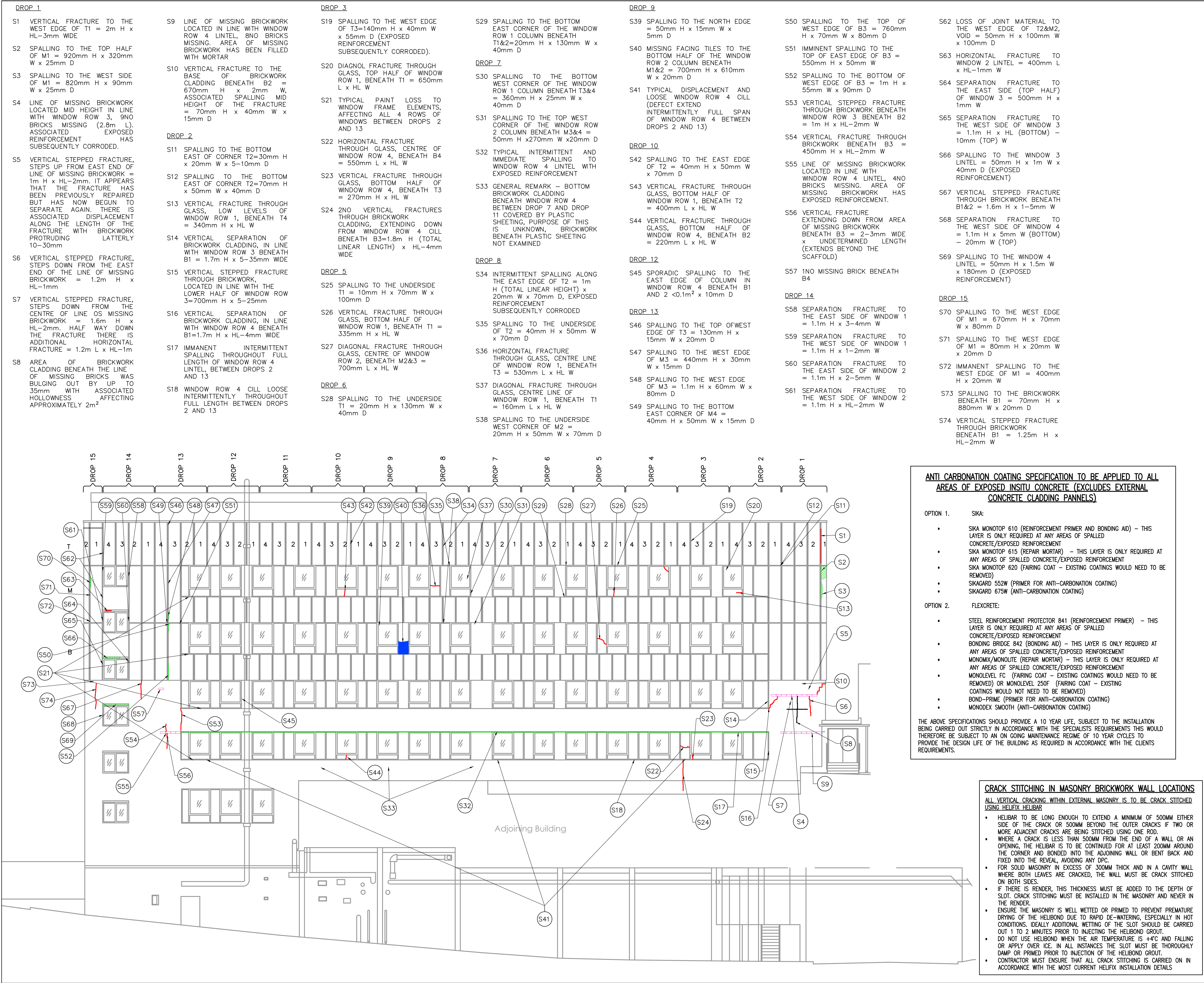
	Single skin/ Cavity wall	Solid Masonry		
		Up to 102.5mm	102.5mm to 225mm	Over 225mm
Depth of slot	25 – 35mm	25 – 40 mm	25 – 40 mm	25 – 40mm On both sides
Vertical Spacing	Every 4 – 6 courses, 300 – 450mm			

Technical Specifications

Material:	Austenitic stainless steel Grade 304 (1.4301) or 316 (1.4401)
Diameter:	6mm (or 4.5mm for thin mortar joints)
Tensile strength (6mm HeliBar):	10 kN
0.2% Proof stress (6mm HeliBar):	900 N/mm ²
Length:	To extend 500mm either side of the crack or outer cracks, if more than one
Standard lengths:	1m, 1.5m and 2m – in packs of 10. May be cut to length on site
Height of slot:	Full height of mortar bed or approx. 10mm if cut through the masonry unit
Bonding agent:	HeliBond cementitious grout. 1 x 3ltr HeliBond = 10 linear metres of crack stitching
RECOMMENDED TOOLING	
For cutting slot up to 40mm deep:	Twin-bladed cutter with vacuum attachment or angle grinder or hammer and mortar chisel
For mixing HeliBond grout:	3-jaw-chuck drill with mixing paddle
For injection of HeliBond into slots:	Helifix Pointing Gun CS with mortar nozzle
For smoothing pointing:	Standard finger trowel
For inserting HeliBar:	HeliBar Insertion Tool

APPENDIX

vii) SHEAR DESIGN ELEVATION DRAWINGS



NOTES

GENERAL

- ALL DIMENSIONS ARE IN MILLIMETRES UNLESS NOTED OTHERWISE.
- ALL LEVELS ARE IN METRES UNLESS NOTED OTHERWISE.
- ALL LEVELS RELATE TO ORDNANCE DATUM UNLESS NOTED OTHERWISE.
- DO NOT SCALE FROM THIS DRAWING. USE FIGURED DIMENSIONS ONLY.
- ANY DISCREPANCIES TO BE REPORTED IMMEDIATELY TO THE ENGINEER.
- THIS DRAWING TO BE READ IN CONJUNCTION WITH ALL RELEVANT ARCHITECTS, SUBCONTRACTORS AND SPECIALISTS DRAWINGS AND SPECIFICATIONS.
- THIS DRAWING IS COPYRIGHT © PROPERTY OF SHEAR DESIGN LIMITED.

INDICATES INDICATIVE AREA OF SPALLING

INDICATES AREA OF MISSING TILES

INDICATES INDICATIVE LINE OF CRACKS AND FRACTURES.

INDICATES THE NUMBER OF MISSING BRICKS

T TOP CLADDING PANEL SECTION

M MIDDLE CLADDING PANEL SECTION

B BOTTOM CLADDING PANEL SECTION

CONCRETE REPAIR TO SPALLING CONCRETE

- ALL SPALLED CONCRETE IS TO BE REMOVED AND CONCRETE REPAIRED USING FOSROC RENDEROC HBS OR FOSROC NITOMORTAR S. ALL SURFACE PREPARATION PRIMING AND INSTALLATION MUST BE CARRIED OUT STRICTLY IN ACCORDANCE WITH FOSROC DESIGN GUIDES AND SPECIFICATIONS.

ANTI CARBONATION COATING SPECIFICATION TO BE APPLIED TO ALL AREAS OF EXPOSED INSITU CONCRETE (EXCLUDES EXTERNAL CONCRETE CLADDING PANELS)

OPTION 1. SIKA:

- SIKA MONOTOP 610 (REINFORCEMENT PRIMER AND BONDING AID) - THIS LAYER IS ONLY REQUIRED AT ANY AREAS OF SPALLED CONCRETE/EXPOSED REINFORCEMENT
- SIKA MONOTOP 615 (REPAIR MORTAR) - THIS LAYER IS ONLY REQUIRED AT ANY AREAS OF SPALLED CONCRETE/EXPOSED REINFORCEMENT
- SIKA MONOTOP 620 (FAIRING COAT) - EXISTING COATINGS WOULD NEED TO BE REMOVED
- SIKAGARD 552W (PRIMER FOR ANTI-CARBONATION COATING)
- SIKAGARD 675W (ANTI-CARBONATION COATING)

OPTION 2. FLEXCRETE:

- STEEL REINFORCEMENT PROTECTOR 841 (REINFORCEMENT PRIMER) - THIS LAYER IS ONLY REQUIRED AT ANY AREAS OF SPALLED CONCRETE/EXPOSED REINFORCEMENT
- BONDING BRIDGE 842 (BONDING AID) - THIS LAYER IS ONLY REQUIRED AT ANY AREAS OF SPALLED CONCRETE/EXPOSED REINFORCEMENT
- MONOMIX/MONOLITE (REPAIR MORTAR) - THIS LAYER IS ONLY REQUIRED AT ANY AREAS OF SPALLED CONCRETE/EXPOSED REINFORCEMENT
- MONOLEVEL FC (FAIRING COAT) - EXISTING COATINGS WOULD NEED TO BE REMOVED OR MONOLEVEL 250F (FAIRING COAT) - EXISTING COATINGS WOULD NOT NEED TO BE REMOVED
- BOND-PRIME (PRIMER FOR ANTI-CARBONATION COATING)
- MONODEX SMOOTH (ANTI-CARBONATION COATING)

THE ABOVE SPECIFICATIONS SHOULD PROVIDE A 10 YEAR LIFE, SUBJECT TO THE INSTALLATION BEING CARRIED OUT STRICTLY IN ACCORDANCE WITH THE SPECIALISTS REQUIREMENTS THIS WOULD THEREFORE BE SUBJECT TO AN ONGOING MAINTENANCE REGIME OF 10 YEAR CYCLES TO PROVIDE THE DESIGN LIFE OF THE BUILDING AS REQUIRED IN ACCORDANCE WITH THE CLIENTS REQUIREMENTS.

CRACK STITCHING IN MASONRY BRICKWORK WALL LOCATIONS

ALL VERTICAL CRACKING WITHIN EXTERNAL MASONRY IS TO BE CRACK STITCHED USING HELIFIX HELIBAR

- HELIFIX TO BE LONG ENOUGH TO EXTEND A MINIMUM OF 500MM EITHER SIDE OF THE CRACK OR 500MM BEYOND THE OUTER CRACKS IF TWO OR MORE ADJACENT CRACKS ARE BEING STITCHED USING ONE ROD.
- WHERE A CRACK IS LESS THAN 500MM FROM THE END OF A WALL OR AN OPENING, THE HELIFIX IS TO BE CONTINUED FOR AT LEAST 200MM AROUND THE CORNER AND BONDED INTO THE ADJOINING WALL OR BENT BACK AND FIXED INTO THE REVEAL, AVOIDING ANY DPC.
- FOR SOLID MASONRY IN EXCESS OF 300MM THICK AND IN A CAVITY WALL WHERE BOTH LEAVES ARE CRACKED, THE WALL MUST BE CRACK STITCHED ON BOTH SIDES.
- IF THERE IS RENDER, THIS THICKNESS MUST BE ADDED TO THE DEPTH OF SLOT. CRACK STITCHING MUST BE INSTALLED IN THE MASONRY AND NEVER IN THE RENDER.
- ENSURE THE MASONRY IS WELL WETTED OR PRIMED TO PREVENT PREMATURE DRYING OF THE HELIBOND DUE TO RAPID DE-WATERING, ESPECIALLY IN HOT CONDITIONS. IDEALLY ADDITIONAL WETTING OF THE SLOT SHOULD BE CARRIED OUT 1 TO 2 MINUTES PRIOR TO INJECTING THE HELIBOND GROUT.
- DO NOT USE THE HELIBOND WHEN THE AIR TEMPERATURE IS +4°C AND FALLING OR APPLY OVER ICE. IN ALL INSTANCES THE SLOT MUST BE THOROUGHLY DAMP OR PRIMED PRIOR TO INJECTION OF THE HELIBOND GROUT.
- CONTRACTOR MUST ENSURE THAT ALL CRACK STITCHING IS CARRIED ON IN ACCORDANCE WITH THE MOST CURRENT HELIFIX INSTALLATION DETAILS

NOTE

THIS DRAWING MUST BE READ IN CONJUNCTION WITH SKYLINE ROPE ACCESS LTD CLADDING INSPECTION REPORT.

GENERAL NOTE - ALL ELEMENTS OF THE SOUTH ELEVATION BENEATH THE LINE OF WINDOW ROW 4 CILL, BETWEEN DROPS 12 AND 15 WERE OBSTRUCTED BY SCAFFOLD AND THEREFORE NOT EXAMINED

REV	DATE	DESCRIPTION	BY	CHK

CLIENT: NEWPORT NORSE

PROJECT: NEWPORT CENTRAL LIBRARY & MUSEUM

TITLE: SOUTH ELEVATION

SHEAR design
Consulting Civil and Structural Engineers

7 Ashtree Court - Woody's Close - Cardiff Gate Business Park - Cardiff - CF23 6RW
Tel: 029 2054 7000 - Fax: 029 2054 7001 - www.shear-design.com - enquiries@shear-design.com

DRAWN	CHECKED	DATE	SCALE
AM	DB	JULY 2016	NTS @ A1

STATUS KEY: I = INFORMATION P = PRELIMINARY A = APPROVAL CO = CONTRACT
T = TENDER C = CONSTRUCTION AB = AS-BUILT

STATUS: 1 DRAWING NUMBER: 16027-002 REVISION: -

DROP 1

- W1 SPALLING TO THE WEST FACE OF T1 = 2.5m H x 330mm W x 10-30mm D
- W2 VERTICAL STEPPED FRACTURE THROUGH BRICKWORK BENEATH B3 = 1.5m H x 1-5mm W
- W3 3NO LOOSE BRICKS, LOCATED 1.5m BENEATH B2
- W4 VERTICAL STEPPED FRACTURE THROUGH BRICKWORK BENEATH B1 = 4.2m H x 1-10mm W
- W5 LINE OF MISSING BRICKWORK, STARTS APPROXIMATELY 3m BENEATH B2 AND EXTENDS INTO DROPS 2&3 AND INTO WINDOW ROW 4 LINTEL. 22NO BRICKS MISSING
- W6 VERTICAL STEPPED FRACTURE THROUGH BRICKWORK, STARTS 3m BENEATH B3 AND EXTENDS DOWN = 1.7m H x 1-5mm W
- W7 TYPICAL SPORADIC JOINT LOSS, MAINLY TO LOWER LEVELS OF BRICKWORK IN LINE WITH WINDOW ROW 4 BETWEEN DROPS 1-3 <0.5M² x 30mm D

DROP 2

- W8 SPALLING TO THE SOUTH EDGE OF T4 = 100mm H x 10mm W x 40mm D
- W9 2NO LOOSE BRICKS, LOCATED 1.5m BENEATH B3&4
- W10 LINE OF JOINT LOSS TO BRICKWORK 1.5m BENEATH B1-4, EXTENDS INTO DROP 1 AND 3 <1M² x 30mm D

DROP 3

- W11 SPALLING TO THE BOTTOM HALF OF THE INFILL BETWEEN T1 AND 2 = 1.1m H x 30mm W x 90mm D
- W12 SPALLING TO THE BOTTOM OF THE WINDOW ROW 2 COLUMN BENEATH M3 = 270mm H x 340mm W x 90mm D
- W13 VERTICAL STEPPED FRACTURE THROUGH BRICKWORK, STARTS 900mm BENEATH B2 AND EXTENDS INTO WINDOW ROW 3 CILL = 1.8m H x 5-10mm W
- W14 IMMANENT INTERMITTENT SPALLING THROUGHOUT FULL LENGTH OF WINDOW ROW 4 LINTEL, BETWEEN DROPS 3 AND 11

DROP 4

- W15 SPALLING TO THE TOP NORTH CORNER OF T2 = 120mm H x 200mm W x 20mm D
- W16 SPALLING TO THE BOTTOM SOUTH CORNER OF T2 = 1.25m H x 250mm W x 30mm D

DROP 5

- W17 2NO VERTICAL STEPPED FRACTURE TO WINDOW ROW 2 COLUMN BENEATH M1 AND 2 = 1.15m H x HL-5mm W
- W18 SPALLING TO WINDOW ROW 2 COLUMN BOTTOM SOUTH CORNER BENEATH M1 AND 2 = 140mm H x 30mm W x 20mm
- W19 SPALLING TO WINDOW ROW 2 COLUMN BOTTOM NORTH CORNER BENEATH M1 AND 2 = 70mm H x 30mm W x 5mm

- W20 SPALLING TO WINDOW ROW 3 COLUMN BOTTOM SOUTH CORNER BENEATH B1 AND 2 = 170mm H x 30mm W x 30mm

- W21 SPALLING TO BRICKWORK JUST ABOVE WINDOW ROW 4 LINTEL BENEATH B4 = 60mm H x 50mm W x 5mm

DROP 6

- W22 TYPICAL PAINT LOSS TO WINDOW FRAME ELEMENTS AFFECTING ALL 4 ROWS OF WINDOWS BETWEEN DROPS 3 AND 11

DROP 7

- W23 SPALLING TO WINDOW ROW 3 COLUMN BOTTOM SOUTH CORNER BENEATH B1 AND 2 = 240mm H x 30mm W x 10mm

- W24 SPALLING TO WINDOW ROW 3 COLUMN BOTTOM NORTH CORNER BENEATH B1 AND 2 = 240mm H x 30mm W x 10mm

- W25 DIAGONAL STEPPED FRACTURE TO WINDOW ROW 2 COLUMN BENEATH B1 AND 2 = 600mm L x HL-2mm W

- W26 TYPICAL SPALLING TO WINDOW ROW 4 LINTEL - AREA WITH EXPOSED REINFORCEMENT (THIS DEFECT EXTENDS INTERMITTENTLY FULL LENGTH OF THE LINTEL BETWEEN DROPS 3 AND 11)

DROP 8

- W27 SPALLING TO THE NORTH EDGE OF T1 = 320mm H x 30mm W x 90mm D

DROP 9

- W28 SPALLING TO WINDOW ROW 2 COLUMN BOTTOM SOUTH CORNER BENEATH M1 AND 2 = 70mm H x 30mm W x 10mm

- W29 SPALLING TO WINDOW ROW 2 COLUMN BOTTOM NORTH CORNER BENEATH M1 AND 2 = 220mm H x 60mm W x 50mm

DROP 10

- W30 SPALLING BETWEEN M3 AND 4 BENEATH WINDOW ROW 2 = 20mm H x 40mm W x 2mm, EXPOSED REINFORCEMENT

DROP 11

- W31 SPALLING TO THE TOP NORTH CORNER OF WINDOW ROW 1 COLUMN BENEATH T1 = 70mm H x 50mm W x 10mm D

- W32 SPALLING TO THE BOTTOM NORTH CORNER OF WINDOW ROW 1 COLUMN BENEATH T1 = 150mm H x 140mm W x 30mm D

- W33 LINE OF JOINT LOSS TO BRICKWORK 1.35m BENEATH B1 AND EXTENDS INTO DROP 12 AND 13 <1M² x 30mm D

- W34 1NO MISSING BRICK, LOCATED 1.35m BENEATH B1

- W35 LINE OF MISSING BRICKWORK 1.35m BENEATH B1 AND EXTENDS INTO DROP 12, 17 NO BRICKS MISSING

- W36 LINE OF JOINT LOSS TO BRICKWORK 3.7m BENEATH B1 AND EXTENDS INTO DROP 12 AND 13

- W37 VERTICAL STEPPED FRACTURE EXTENDING UP AND TO THE NORTH FROM THE CILL EDGE WINDOW ROW4 BENEATH B1. THE FRACTURE EXTENDS INTO THE LINE OF MISSING BRICKWORK AT 1.35m BENEATH THE BOTTOM EDGE OF CONCRETE CLADDING AT DROP 12. TOTAL LINEAR LENGTH OF THE FRACTURE = 4.5m AND IS HL-3mm WIDE

DROP 12

- W38 SPALLING TO THE BOTTOM NORTH CORNER OF T1 = 30mm H x 100mm W x 50mm D

- W39 SPALLING TO THE TOP SOUTH CORNER OF T3 = 500mm H x 450mm W x 40mm D

- W40 SPALLING TO THE TOP SOUTH CORNER OF T3 = 500mm H x 450mm W x 40mm D

- W41 VERTICAL FRACTURE THROUGH BRICKWORK LOCATED 4m BENEATH B3 = 500mm H x 450mm W x 40mm D

DROP 13

- W42 2NO SPALL REPAIRS TO THE TOP OF B1 AND 2, EACH = 100mm H x 100mm W

- W43 SPALL REPAIR TO SOUTH OF B1 = 230mm H x 150mm W

- W44 2NO MISSING BRICKS TO BRICKWORK CLADDING BENEATH B1 AND B4

- W45 VERTICAL FRACTURE THROUGH BRICKWORK STARTED 3m BENEATH B2 AND 3 AND EXTENDED DOWNWARDS 1.65m, OPEN 1-3mm

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- OPTION 1. SIKA:
- SIKA MONOTOP 610 (REINFORCEMENT PRIMER AND BONDING AID) - THIS LAYER IS ONLY REQUIRED AT ANY AREAS OF SPALLED CONCRETE/EXPOSED REINFORCEMENT
 - SIKA MONOTOP 615 (REPAIR MORTAR) - THIS LAYER IS ONLY REQUIRED AT ANY AREAS OF SPALLED CONCRETE/EXPOSED REINFORCEMENT
 - SIKA MONOTOP 620 (FAIRING COAT - EXISTING COATINGS WOULD NEED TO BE REMOVED)
 - SIKAGARD 552W (PRIMER FOR ANTI-CARBONATION COATING)
 - SIKAGARD 675W (ANTI-CARBONATION COATING)
- OPTION 2. FLEXCRETE:
- STEEL REINFORCEMENT PROTECTOR 841 (REINFORCEMENT PRIMER) - THIS LAYER IS ONLY REQUIRED AT ANY AREAS OF SPALLED CONCRETE/EXPOSED REINFORCEMENT
 - BONDING BRIDGE 842 (BONDING AID) - THIS LAYER IS ONLY REQUIRED AT ANY AREAS OF SPALLED CONCRETE/EXPOSED REINFORCEMENT
 - MONOMIX/MONOLITE (REPAIR MORTAR) - THIS LAYER IS ONLY REQUIRED AT ANY AREAS OF SPALLED CONCRETE/EXPOSED REINFORCEMENT
 - MONOLEVEL FC (FAIRING COAT - EXISTING COATINGS WOULD NEED TO BE REMOVED) OR MONOLEVEL 250F (FAIRING COAT - EXISTING COATINGS WOULD NOT NEED TO BE REMOVED)
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THE ABOVE SPECIFICATIONS SHOULD PROVIDE A 10 YEAR LIFE, SUBJECT TO THE INSTALLATION BEING CARRIED OUT STRICTLY IN ACCORDANCE WITH THE SPECIALISTS REQUIREMENTS THIS WOULD THEREFORE BE SUBJECT TO AN ON GOING MAINTENANCE REGIME OF 10 YEAR CYCLES TO PROVIDE THE DESIGN LIFE OF THE BUILDING AS REQUIRED IN ACCORDANCE WITH THE CLIENTS REQUIREMENTS.

CRACK STITCHING IN MASONRY BRICKWORK WALL LOCATIONS

- ALL VERTICAL CRACKING WITHIN EXTERNAL MASONRY IS TO BE CRACK STITCHED USING HELIFIX HELIBAR
- HELIBAR TO BE LONG ENOUGH TO EXTEND A MINIMUM OF 500MM EITHER SIDE OF THE CRACK OR 500MM BEYOND THE OUTER CRACKS IF TWO OR MORE ADJACENT CRACKS ARE BEING STITCHED USING ONE ROD.
 - WHERE A CRACK IS LESS THAN 500MM FROM THE END OF A WALL OR AN OPENING, THE HELIBAR IS TO BE CONTINUED FOR AT LEAST 200MM AROUND THE CORNER AND BONDED INTO THE ADJOINING WALL OR BENT BACK AND FIXED INTO THE REVEAL, AVOIDING ANY DPC.
 - FOR SOLID MASONRY IN EXCESS OF 300MM THICK AND IN A CAVITY WALL WHERE BOTH LEAVES ARE CRACKED, THE WALL MUST BE CRACK STITCHED ON BOTH SIDES.
 - IF THERE IS RENDER, THIS THICKNESS MUST BE ADDED TO THE DEPTH OF SLOT. CRACK STITCHING MUST BE INSTALLED IN THE MASONRY AND NEVER IN THE RENDER.
 - ENSURE THE MASONRY IS WELL WETTED OR PRIMED TO PREVENT PREMATURE DRYING OF THE HELIBOND DUE TO RAPID DE-WATERING, ESPECIALLY IN HOT CONDITIONS. IDEALLY ADDITIONAL WETTING OF THE SLOT SHOULD BE CARRIED OUT 1 TO 2 MINUTES PRIOR TO INJECTING THE HELIBOND GROUT.
 - DO NOT USE HELIBOND WHEN THE AIR TEMPERATURE IS +4°C AND FALLING OR APPLY OVER ICE. IN ALL INSTANCES THE SLOT MUST BE THOROUGHLY DAMP OR PRIMED PRIOR TO INJECTION OF THE HELIBOND GROUT.
 - CONTRACTOR MUST ENSURE THAT ALL CRACK STITCHING IS CARRIED ON IN ACCORDANCE WITH THE MOST CURRENT HELIFIX INSTALLATION DETAILS

CONCRETE REPAIR TO SPALLING CONCRETE

- ALL SPALLED CONCRETE IS TO BE REMOVED AND CONCRETE REPAIRED USING FOSROC RENDEROC HBS OR FOSROC NITOMORTAR S. ALL SURFACE PREPARATION PRIMING AND INSTALLATION MUST BE CARRIED OUT STRICTLY IN ACCORDANCE WITH FOSROC DESIGN GUIDES AND SPECIFICATIONS.



WEST ELEVATION

NOTES

GENERAL

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- INDICATES THE NUMBER OF MISSING BRICKS
- T TOP CLADDING PANEL SECTION
- M MIDDLE CLADDING PANEL SECTION
- B BOTTOM CLADDING PANEL SECTION

NOTE

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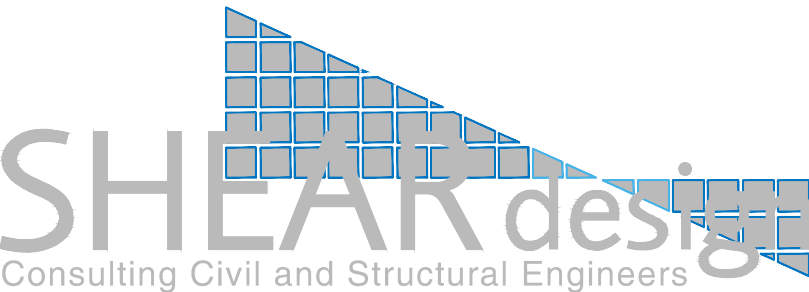
REV	DATE	DESCRIPTION	BY	CHK
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AMENDMENTS

CLIENT: NEWPORT NORSE

PROJECT: NEWPORT CENTRAL LIBRARY & MUSEUM

TITLE: WEST ELEVATION

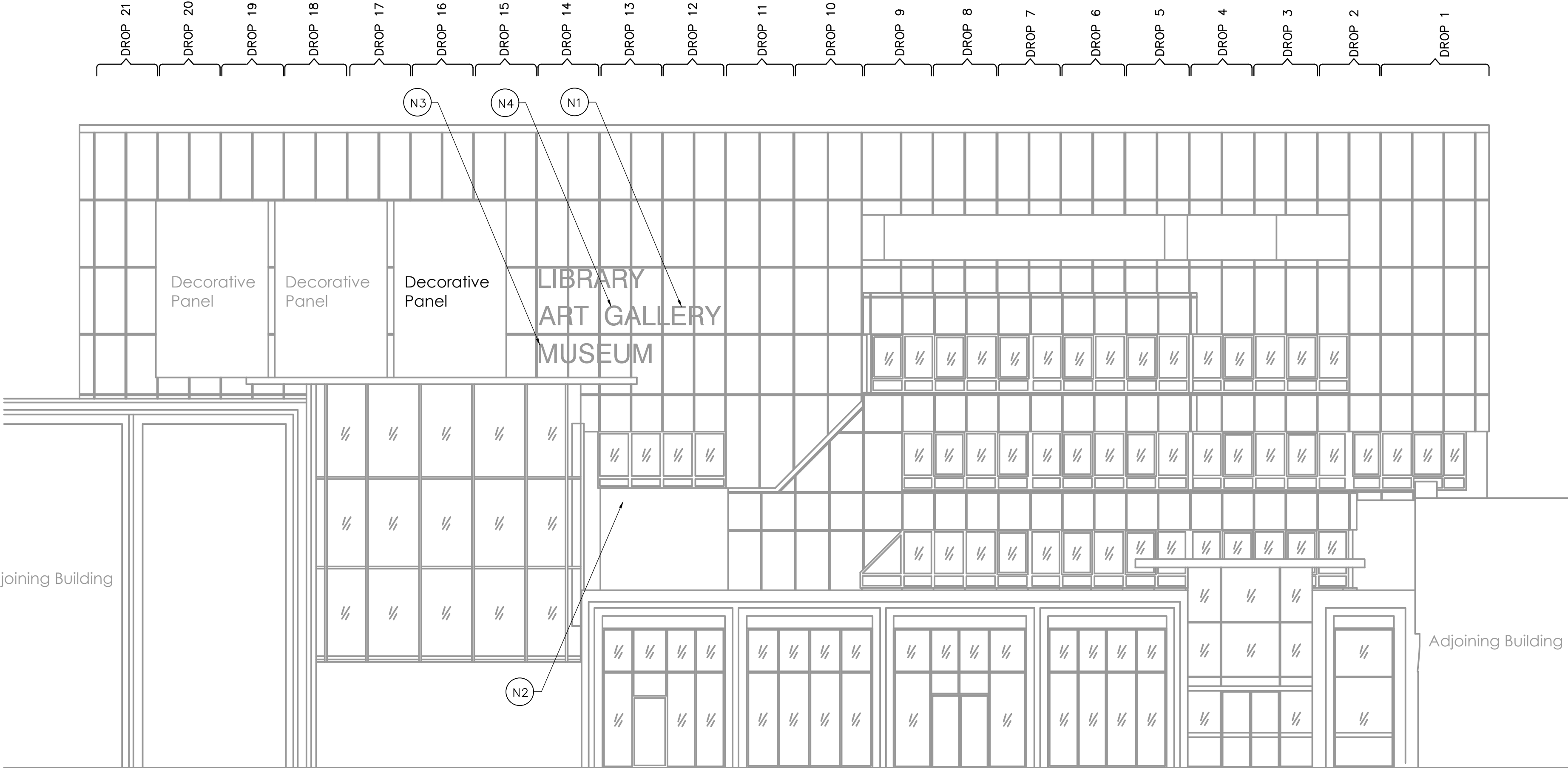


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DRAWN: AM CHECKED: DB DATE: JULY 2016 SCALE: NTS @ A1

STATUS KEY: I = INFORMATION P = PRELIMINARY A = APPROVAL CO = CONTRACT
T = TENDER C = CONSTRUCTION AB = AS-BUILT

STATUS: DRAWING NUMBER: 16027-003 REVISION: -



NORTH ELEVATION

DROP 12

- N1 LOOSE LETTER "E" LOCATED ON A 3RD PANEL DOWN
- N2 VERTICAL FRACTURE THROUGH BRICKWORK BENEATH ALUMINUM PANELS = 680mm x HL-2mm

DROP 13

- N3 LOOSE LETTER "M" LOCATED ON A 4TH PANEL DOWN
- N4 LOOSE LETTER "G" LOCATED ON A 3RD PANEL DOWN

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REV	DATE	DESCRIPTION	BY	CHK
AMENDMENTS				

CLIENT: NEWPORT NORSE

PROJECT: NEWPORT CENTRAL LIBRARY & MUSEUM

TITLE: NORTH ELEVATION

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AM	DB	JULY 2016	NTS @ A1
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I	16027-004	-	

DROP 1

- E1 SPALLING TO THE NORTH EDGE OF T4= 50mm H x 15mm W x 5mm D
- E2 SPALLING TO THE SOUTH EDGE OF M1 = 50mm H x 15mm W x 10mm D
- E3 SPALLING TO THE SOUTH EDGE OF M1 = 30mm H x 15mm W x 10mm D
- E4 SPALLING TO THE UNDERSIDE OF T1&T2 = 10mm H x 150mm W x 100mm D

DROP 2

- E5 SPALLING TO THE TOP NORTH HALF OF M4 = 2.2m H x 320mm W x 5-25mm D
- E6 IMMINENT SPALLING TO THE BOTTOM NORTH HALF OF M4 = 2.5m H x 320mm W
- E7 SPALLING TO THE BOTTOM SOUTH CORNER OF M3 = 30mm H x 80mm W x 60mm D

DROP 3

- E8 SPALLING TO THE NORTH EDGE OF T2 = 500mm H x 50mm W x 50mm D (EXPOSED REINFORCEMENT SUBSEQUENTLY CORRODED)
- E9 SPALLING TO THE BOTTOM EDGE OF M4 = 30mm H x 100mm W x 10mm D

DROP 7

- E10 SPALLING TO THE TOP NORTH CORNER OF M3 = 90mm H x 70mm W x 5-30mm D

DROP 8

- E11 SPALLING TO THE TOP SOUTH CORNER OF M3 = 30mm H x 30mm W x 20mm D
- E12 SPALLING TO THE TOP SOUTH CORNER OF M1 = 30mm H x 20mm W x 20mm D

DROP 10

- E13 SPALLING TO THE EDGE OF T1 = 60mm H x 20mm W x 20mm D

DROP 11

- E14 TYPICAL PAINT LOSS TO WINDOW ELEMENTS
- E15 GENERAL VIEW OF BRICKWORK CLADDING BENEATH DROP 11 - APPEARS TO HAVE BEEN RECENTLY REPAINTED

DROP 12

- E16 SPALLING TO THE BOTTOM NORTH CORNER OF T1 = 70mm H x 70mm W x 70mm D

DROP 13

- E17 SPALLING TO THE TOP OF M3 = 580mm H x 20mm D (EXTENDS INTO DROP 1 - SOUTH ELEVATION)
- E18 VERTICAL FRACTURE TO THE NORTH EDGE OF T3 = 2.5m H x HL-5mm WIDE
- E19 GENERAL VIEW OF BRICKWORK CLADDING BENEATH DROP 13 SHOWING VERTICAL FRACTURE, LOCATED 90mm IN FROM SOUTH EDGE OF EAST ELEVATION = 2.2m H x HL-2mm W

BRICKWORK CLADDING BENEATH OVERHANG

BOTTOM HALF - FROM 2m DOWN TO THE FLOOR LEVEL

- E20 SPALLING TO THE EAST EDGE OF SOUTH RETURN, TOP OF ACCESS DOOR = 140mm H x 40mm W x 20mm D
- E21 POOR CONCRETE MIX AT THE MITRE BETWEEN SOUTH RETURN AND EAST ELEVATION
- E22 SPORADIC AREAS OF HONEYCOMBING THROUGHOUT CONCRETE SECTION SEPARATING RED AND BLUE BRICKS
- E23 SPORADIC AREAS OF SPALLING THROUGHOUT THE LINE OF CONCRETE <0.5m² x 10-30mm D

- E24 SPALLING TO THE CONCRETE SECTION, LOCATED 20m FROM THE SOUTH RETURN, 2m ABOVE FLOOR LEVEL = 70mm H x 180mm W x 30mm D

- E25 VERTICAL FRACTURE THROUGH CONCRETE SECTION, LOCATED 19.5m FROM THE SOUTH RETURN, 1.8m ABOVE FLOOR LEVEL = 360mm H x HL-2mm W

- E26 POOR CONCRETE MIX PILLAR LOCATED 27.9m FROM SOUTH RETURN

TOP HALF - FROM 2m UP TO THE UNDERSIDE OF OVERHANG

- E27 TYPICAL JOINT LOSS THROUGHOUT MID HEIGHT BRICKWORK CLADDING, AFFECTING <0.5m² IN TOTAL TO A MAXIMUM DEPTH OF 20mm

- E28 SPALLING TO BRICKWORK LOCATED 2.5m FROM SOUTH RETURN, 2.3m ABOVE FLOOR LEVEL = 60mm H x 100mm W x 10mm D

- E29 VERTICAL FRACTURE THROUGH RENDER SECTION, LOCATED 9.9m FROM THE SOUTH RETURN = 200mm H x 2mm W

- E30 VERTICAL FRACTURE THROUGH RENDER SECTION, LOCATED 10.7m FROM THE SOUTH RETURN = 1m H x HL-3mm W

- E31 VERTICAL FRACTURE THROUGH RENDER SECTION, LOCATED 12.8m FROM THE SOUTH RETURN = 1m H x HL-1mm W

- E32 VERTICAL FRACTURE THROUGH RENDER SECTION, LOCATED 16.5m FROM THE SOUTH RETURN = 1m H x HL-1mm W

- E33 3NO FRACTURES THROUGH RENDER SECTION, LOCATED 26.5m FROM THE SOUTH RETURN = 2m TOTAL LINEAR LENGTH x HL-1mm W

- E34 VERTICAL STEPPED FRACTURE THROUGH BRICKWORK, LOCATED 27.6m FROM THE SOUTH RETURN, 2.2m ABOVE GROUND LEVEL = 300mm H x 5mm W

- E35 3NO FRACTURES THROUGH RENDER SECTION, LOCATED 28.2m FROM THE SOUTH RETURN = 3.5m TOTAL LINEAR LENGTH x HL-1mm W

- E36 MISSING BRICKWORK TO THE SOUTH CORNER, LOCATED 2.9m ABOVE FLOOR LEVEL = 70mm H x 220mm W x 100mm D

- E37 TYPICAL SPORADIC SPALLING THROUGHOUT <0.1m² IN TOTAL TO A MAXIMUM DEPTH OF 20mm

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CRACK STITCHING IN MASONRY BRICKWORK WALL LOCATIONS

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- IF THERE IS RENDER, THIS THICKNESS MUST BE ADDED TO THE DEPTH OF SLOT. CRACK STITCHING MUST BE INSTALLED IN THE MASONRY AND NEVER IN THE RENDER.
- ENSURE THE MASONRY IS WELL WETTED OR PRIMED TO PREVENT PREMATURE DRYING OF THE HELIBOND DUE TO RAPID DE-WATERING, ESPECIALLY IN HOT CONDITIONS. IDEALLY ADDITIONAL WETTING OF THE SLOT SHOULD BE CARRIED OUT 1 TO 2 MINUTES PRIOR TO INJECTING THE HELIBOND GROUT.
- DO NOT USE HELIBOND WHEN THE AIR TEMPERATURE IS +4°C AND FALLING OR APPLY OVER ICE. IN ALL INSTANCES THE SLOT MUST BE THOROUGHLY DAMP OR PRIMED PRIOR TO INJECTION OF THE HELIBOND GROUT.
- CONTRACTOR MUST ENSURE THAT ALL CRACK STITCHING IS CARRIED ON IN ACCORDANCE WITH THE MOST CURRENT HELIFIX INSTALLATION DETAILS

NOTES

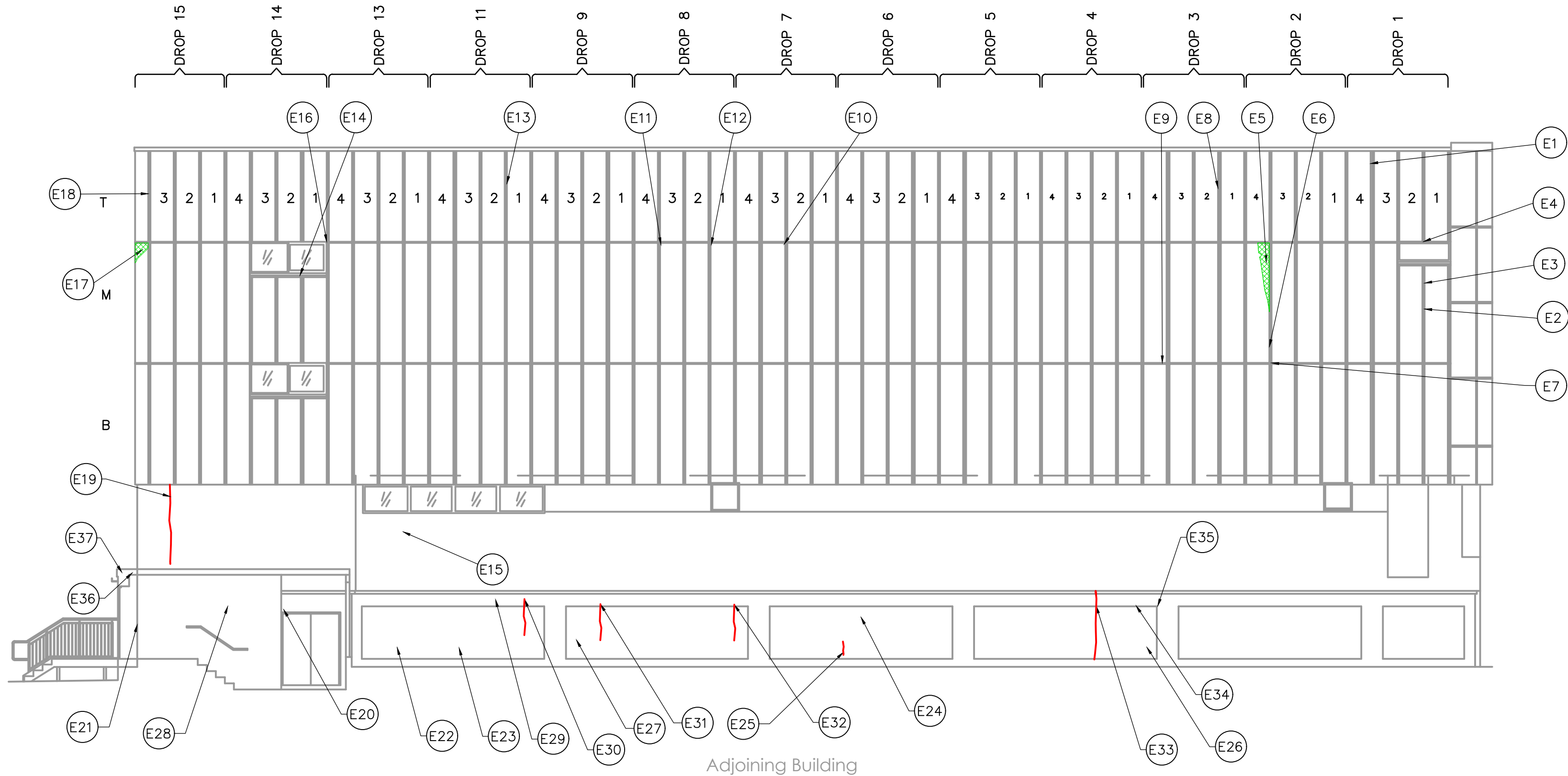
GENERAL

- ALL DIMENSIONS ARE IN MILLIMETRES UNLESS NOTED OTHERWISE.
- ALL LEVELS ARE IN METRES UNLESS NOTED OTHERWISE.
- DO NOT SCALE FROM THIS DRAWING. USE FIGURED DIMENSIONS ONLY.
- ANY DISCREPANCIES TO BE REPORTED IMMEDIATELY TO THE ENGINEER.
- THIS DRAWING TO BE READ IN CONJUNCTION WITH ALL RELEVANT ARCHITECTS, SUBCONTRACTORS AND SPECIALISTS DRAWINGS AND SPECIFICATIONS.
- THIS DRAWING IS COPYRIGHT © PROPERTY OF SHEAR DESIGN LIMITED.

- INDICATES INDICATIVE AREA OF SPALLING
- INDICATES AREA OF MISSING TILES
- INDICATES INDICATIVE LINE OF CRACKS AND FRACTURES.
- INDICATES THE NUMBER OF MISSING BRICKS
- T TOP CLADDING PANEL SECTION
- M MIDDLE CLADDING PANEL SECTION
- B BOTTOM CLADDING PANEL SECTION

CONCRETE REPAIR TO SPALLING CONCRETE

- ALL SPALLED CONCRETE IS TO BE REMOVED AND CONCRETE REPAIRED USING FOSROC RENDEROC HBS OR FOSROC NITOMORTAR S. ALL SURFACE PREPARATION PRIMING AND INSTALLATION MUST BE CARRIED OUT STRICTLY IN ACCORDANCE WITH FOSROC DESIGN GUIDES AND SPECIFICATIONS.



The tunnel continues towards shopping centre loading bay

EAST ELEVATION

NOTE

THIS DRAWING MUST BE READ IN CONJUNCTION WITH SKYLINE ROPE ACCESS LTD CLADDING INSPECTION REPORT.

REV	DATE	DESCRIPTION	BY	CHK
AMENDMENTS				

CLIENT: NEWPORT NORSE

PROJECT: NEWPORT CENTRAL LIBRARY & MUSEUM

TITLE: EAST ELEVATION

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DRAWN	CHECKED	DATE	SCALE
AM	DB	JULY 2016	NTS @ A1
STATUS KEY: I = INFORMATION P = PRELIMINARY A = APPROVAL CO = CONTRACT			
T = TENDER C = CONSTRUCTION AB = AS-BUILT			
STATUS	DRAWING NUMBER	REVISION	
I	16027-001	-	