Appendix I
Joint Report: Slab Testing

COVEY
ASSOCIATES PTY LTD
Consulting Engineers

JOINT REPORT
CARBONATION TESTING - SUSPENDED SLAB

AT

PAVILION 1, 50 MARINE PARADE, REDCLIFFE

BY

ACOR & COVEY

FOR

MORETON BAY REGIONAL COUNCIL

PROJECT NO: 223164
REF: 27735RPT - A
DATE: 22 DECEMBER 2022

DOCUMENT ISSUE APPROVAL

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Title:	Carbonation Testing – Suspended Slab – Pavilion 1
Client:	Moreton Bay Regional Council
Date:	22 December 2022
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APPENDIX B	Summary of Carbonation Results

1.0 BACKGROUND

A greater spread of test locations was deemed to be required.

2.0 METHODOLOGY

On 10 November 2022, 11 Core holes were drilled in the suspended slab of Pavilion 1 for the purpose of carbonation testing. This was carried out by the senior lead engineer from ACOR in the presence of a senior structural engineer from Covey.

50mm diameter cores were cut, instantly dried using clean cloths then immediately tested with phenolphthalein, measured, recorded and photographed. These samples were kept for possibility of future testing if required.

The location of the core holes are as per the plan in Appendix A and the summary of the results is tabulated in Table 1 of Appendix B attached. The photos and descriptions of each cored sample are listed in section 3 below.

Further onsite checks measuring cover to bottom steel reinforcement from the soffit of the slab were undertaken by the senior engineer from Covey and are identified in Appendix A attached. Most of these locations were at the areas of steel exposed from the previous tests.

3.0 TEST RESULTS

<u>Core C1</u>
This first core was taken above the undercroft area located between grids 2-3:H-R. It is thought that this area was originally slab on ground and then dug out to put pipework in for toilets post circa 1975 to 2000. Note the uneven slab soffit with the same red soil on it as the ground beneath.

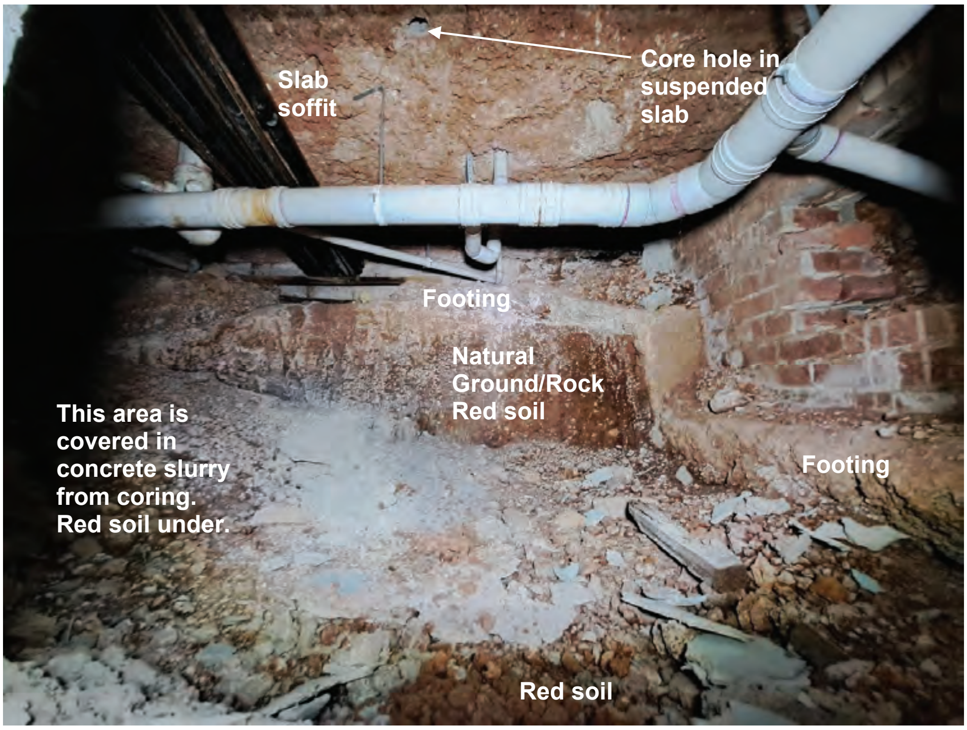


Image 1: Soffit of suspended slab above undercroft area.

Image 2: Location of C1

Image 3: Core at C1 with tape measure from soffit to top.

Core C2

This core was taken in the area of slab that was originally enclosed upstairs above the female bathrooms.

Image 4: Location of C2

Image 5: Core at C2 with tape measure starting at 0 at the top of the slab. Note another 10-15mm topping/grout above original slab. This topping/grout appears to be and is assumed to be pre 1975.Note the carbonation from top and bottom of the slab.

Image 6: C2 Core with bottom
Transverse steel bar at 31 cover

Image 7: C2 Core with
Bottom Steel at 25 cover

Core C3

This core was also taken in the area of slab that was originally enclosed upstairs above the mens bathrooms. This Core was fully carbonated so another core was taken approximately one metre away (labelled 3x).

Image 8: Location of C3 and C3x

Image 9: Core C3 with tape measuring from top of slab to soffit – Fully Carbonated

Core C3x

This core was taken approximately 1m away from the previous core towards grid R. But it also showed a fully carbonated result.

Image 10: Core C3x with tape measuring from top of slab to soffit – Also fully Carbonated. Note poor compaction and boney concrete.

Core C4

This Core was taken in an area that was originally the tiled balcony slab that must have had a sandy grout layer under the tiles that varied in thickness. This area was around 75-110mm lower than the internal upstairs area. (there was a 75mm step down detailed on the original blueprints). This whole area later had a topping slab added, this has been denoted topping* in the table of results.

Image 11: Location of C4 and C3x

Image 12: Core C4 with tape measuring from top of slab to soffit

Image 13: C4 Core with
20 to 30mm carbonation

Image 14: C4 Core with
20-30mm Carbonation

<u>Core C4x</u>
This core was taken in between Cores C4 and C5.

Image 15: Location of C4 and C4x

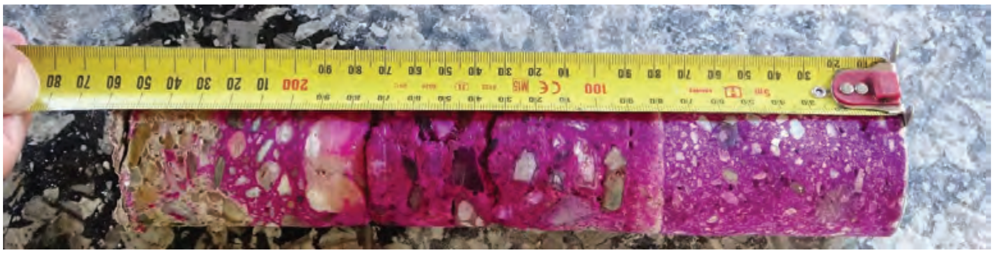


Image 16: Core C4 with tape measuring from top of slab to soffit – Note topping slab, then balcony grout topping then original slab which also has a rendered and painted soffit.

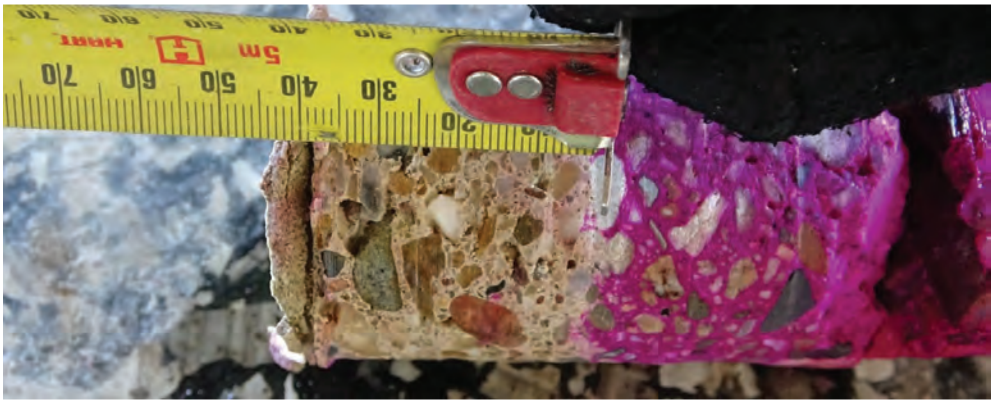


Image 17: Core C4x with tape measuring 40mm carbonation from soffit.

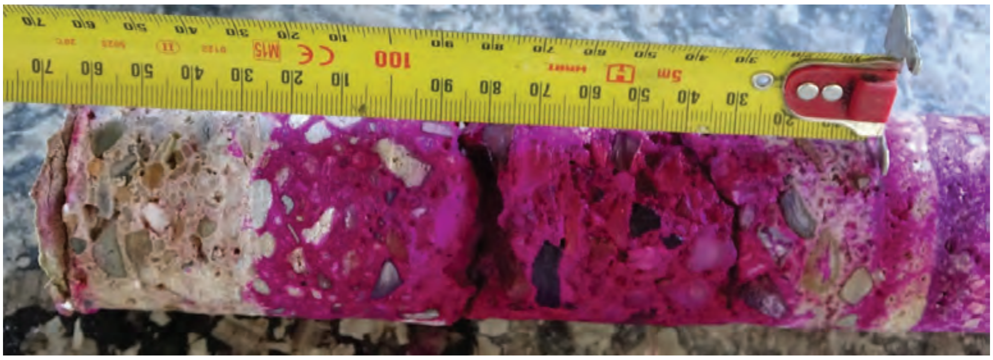


Image 18: Core C4x with tape measuring 165 original slab and 40mm carbonation from soffit.

Core C5

This core was taken near grid Q and approximately 1.3m off grid 5 towards grid 4. This area also used to be the original tiled balcony set down from the upstairs internal area similarly to C4 and C4x above.

Image 19: Location of C5

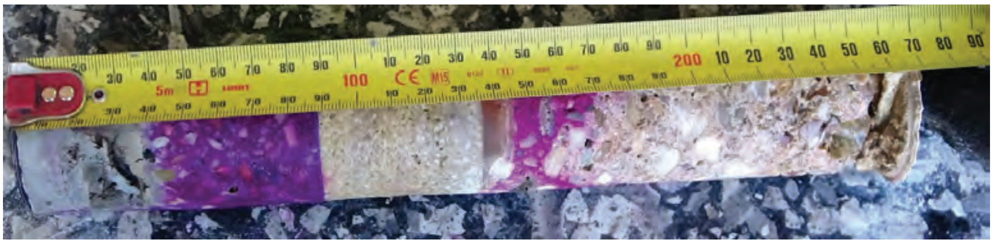


Image 20: Core C5 with tape measuring from the top of the slab to the soffit.

Core C6

This core was taken in behind the bar near grid F between grids 3 and 4. This area also used to be the original tiled balcony on the Southwestern side.

Image 21: Location of C6

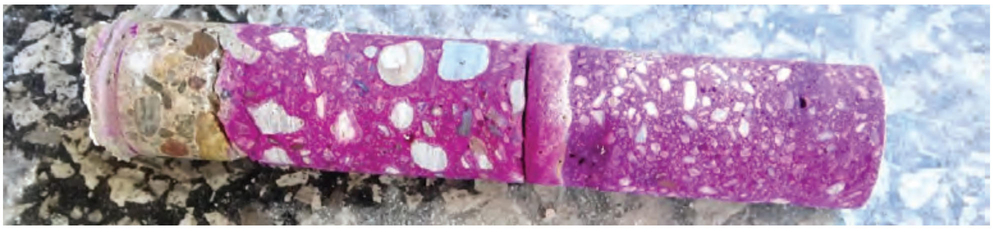


Image 22: Core C6 with top of topping slab to the right then moving left there is the sanding tile grout then original suspended slab with render and paint on the soffit at the left end of the above sample.

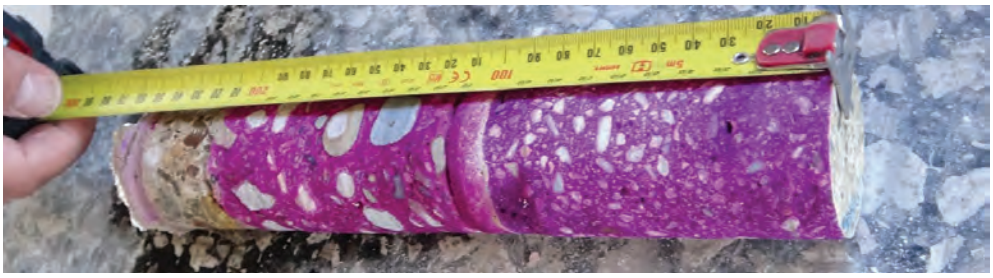


Image 23: Core C6 with tape measuring from the top of the slab to the soffit

Image 24: Core C6 with tape measuring 40mm of carbonation from the soffit of the slab

Core C7

This core was taken in The North Eastern Corner of the suspended slab near grid W:5. This was originally the external corner of the tiled balcony.

Image 25: Location of C7 and C8

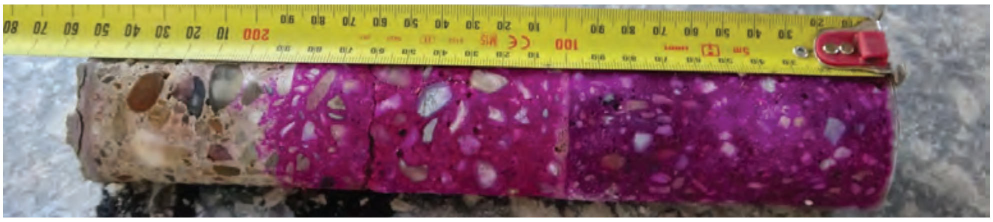


Image 26: Core C7 with tape measuring from the top of the slab to the soffit

Core C8

This core was taken near grid U:3-4 as shown in image 24 above. This was originally the external tiled balcony above the mens changerooms and bathrooms.

Image 26: Core C8 with tape measuring from the top of the slab to the soffit

Image 27: Core C8 with tape measuring from the slab soffit to the centreline of the bottom steel reo

Core C9

This core was taken at grid M:7 in the original external tiled balcony above the entrance.

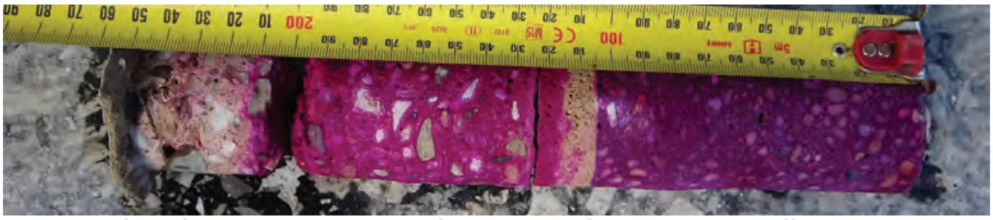


Image 28: Core C9 with tape measuring from the top of the slab to the soffit

The above image shows carbonation in the original tile grout at around 105mm depth to 115mm as well as carbonation from the underside of the suspended slab approximately40-45mm from the soffit.

4.0 DISCUSSION AND CONCLUSION

The site investigation on carbonation depth of the suspended slab as presented in the previous section, indicates an average carbonation depth of about 35-40mm from soffit of the slab. There are several locations (including core hole C5 and C7) where the carbonation depth was higher and in two locations (core sample C3 and c3x) carbonation was full depth. The average concrete cover to bottom reinforcement of the slab was measured previously to be about 20-40mm and in this investigation the cover measured ranged from 15-45mm.

Comparing the results of the carbonation depth testing with the current concrete cover to the bottom reinforcement of the slab would confirm that the almost the entire soffit of the slab has been carbonated to the depth of bottom reinforcement. This indicates that the protective passive layer provided by the concrete to the bottom reinforcement of the slab is already gone and at some locations the corrosion to the steel has started. The corrosion to the bottom steel at some locations is quite aggressive and has developed to an extent that the concrete cover over the steel is spalling. There are several areas of the soffit of the slab that have not shown any sign of concrete cracking or spalling which could suggest the corrosion of steel has not yet started or developed to an extent to cause concrete cracking or concrete spalling.

On the other hand, the initial site investigation by BG&E confirmed the level of the chloride concentration at the majority of the core samples from the soffit of the slab are almost at the critical chloride thresholds (ranging between 0.03 and 0.06) at the reinforcement depth from the soffit of the slab. This also confirms that the protective passive layer to the bottom reinforcement of almost the entire slab is affected by chloride attack to an extent where corrosion to steel reinforcement has been advanced in some locations.

The carbonation depth at top of the slab was also measured at all core samples locations as presented in the previous section. The test results indicate the carbonation depth from the top of the slab is ranging from 5-60mm (whether in the concrete topping or within the structural slab) and at two locations (C3 and C3x) is full depth. As the previous investigation by BG&E confirmed, there is no reinforcement to the top of the slab, therefore the carbonation depth at top of the slab would not be of any concern. It should be however mentioned that during ACOR investigation on site, a simple scanner (only one available) was used which indicated some top reinforcement over the beam along grid 4 in the central area of the building. However, the GPR scanner used by BG&E has a much higher accuracy so, we can assume there is no top reinforcement to the slab and assess the ultimate capacity of the slab based on the bottom reinforcement only. Regardless of whether the slab is single span or continuous span it still needs strengthening (refer to the BG&E slab analysis - see report in Appendix K of Main Report).

5.0 LIMITATIONS

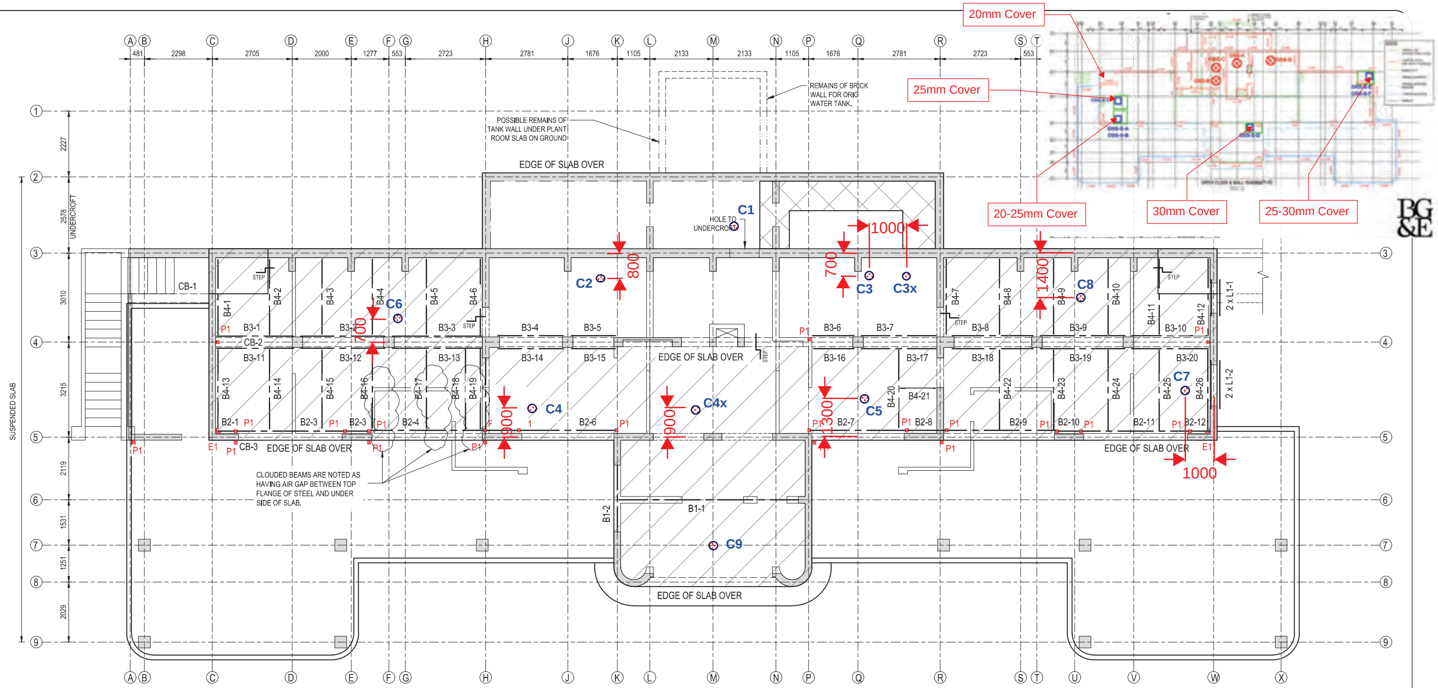
The opinions, conclusions and any recommendations in this report are based on information from, and testing undertaken at or in connection with, specific sample points. Site conditions in other parts of the site may be different to those found at the specific sample points.

The opinions, conclusions and recommendations in this report are based on the assumptions made by Covey Associates and ACOR described in this report. Covey Associates and ACOR disclaim any liability arising from any of these assumptions being incorrect.

The opinions, conclusions and any recommendations in this report are based on the conditions encountered and information reviewed at the date of preparation of the report. Covey Associates and ACOR have no responsibility or obligation to update this report to account for events or changes subsequent to the date the report was prepared.

APPENDIX A

LOCATION OF CORE HOLES
AND
COVER TO STEEL REINFORCEMENT



CORE LOCATION PLAN

UPPER FLOOR SLAB FRAMING PLAN

SCALE A1 - 1:62.5
SCALE A3 - 1:125

C9 APPROXIMATE CORE LOCATIONS

700 APPROXIMATE DISTANCE TO GRID

15mm Cover COVER TO REO MEASURED AT BREAKOUTS

STRUCTURAL ELEMENT - IDENTIFICATION			
ID	DESCRIPTION	MATERIAL	ORIGIN
P1	100 SHS	STEEL	POST 2000
EA1	150x10 EA	STEEL	POST 2000
B1	360UB 50.7	STEEL	POST 2000
B2	250PFC	STEEL	POST 2000
B3	200 PFC	STEEL	POST 2000
B4	180 PFC	STEEL	POST 2000
B5	150PFC	STEEL	POST 2000
L1	100EA x 6	STEEL	POST 2000
CB-1	GRID A - C	CONCRETE	ORIGINAL
CB-2	GRID C - W	CONCRETE	ORIGINAL
CB-3	GRID C - W	CONCRETE	

NOTE:

ALL DIMENSION ARE BASED OFF SCANNED PDF FROM 1937 ARCHITECTURAL SKETCHES, ROUGH SITE MEASURES. DIMENSIONS ARE APPROXIMATELY +/- 100mm
FOR MORE ACCURATE PLANS SEE DRAWINGS PRODUCED BY 3D CLOUD POINT (BY OTHERS).

ACOR Consultants (QLD) Pty Ltd

LEGEND

- ORIGINAL 1937 ARCHITECTURAL LAYOUT
- CURRENT LAYOUT NOM. 100 mm TOLERANCE
- BEAMS OVER
- BEAMS UNDER
- UNDERCROFT WALLS
- CURRENT LOWER SUPPORTING WALLS / COL.

- HATCHED AREA INDICATES TOPPING SLAB OVER ORIGINAL SLAB
- 100mm HIGH CONCRETE PLINTH OVER ORIGINAL SLAB
- STEP — ORIGINAL STEP PRIOR TO TOPPING

Check all dimensions before commencement of work. Check Site boundary dimensions from the Title plans Check Building Boundary clearances by set out. Notwithstanding the information supplied on this drawing, the location, depth and extent of underground or overhead services are to be confirmed and protected on site by the contractor prior to commencement of works.

Checked – RG
Design – CJT Drawn – MS
Scales – AS SHOWN
Document Stage – INFORMATION

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Approved

Drawing title – PAVILLION 1 UPPER FLOOR FRAMING STRUCTURE

Project – SUTTONS BEACH PAVILIONS

Client – MORETON BAY REGIONAL COUNCIL

Site – SUTTONS BEACH, REDCLIFFE, QLD, 4021

Project No. 223164 Sheet No. SK102

Digital Ref : 223164_SK102 FLOOR FRAMING STRUCTURE Issue : E

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Plot 04 Nov, 2022 - 09:25

APPENDIX B

SUMMARY OF CARBONATION RESULTS

Table 1 – Summary of results

No.	Location	Total Sample length (mm)	Topping Slab depth (mm)	Original Slab Depth (mm)	Carbonation from top (mm)	Carbonation from bottom (mm)	Depth of reo from btm (mm)	Cover to reo (mm)	Comments
C1	Near Grids M:3	210-220	5-10	210-220	15-20	40	40-50	36-46	Underside of slab very rough and uneven
C2	Near Grids K:3-4	145	10	145	50-60	35-40	28-35	31	BTM Bar is in carbonation zone but not corroded.
C3	Near Grids Q:3-4	155	10	155	Fully Carbonated	Fully Carbonated			
C3x	Nearer Grids R:3-4	160	15	160	Fully Carbonated	Fully Carbonated			
C4	Near Grids H-J:4-5	250	95* + 5	150	5	20-30			
C4x	Near Grids M:5	260	80* + 10	165	0	40			
C5	Near Grids Q:5	270	85* + 45	135	0-40 in topping* All of original grout topping	60-100			
C6	Near Grids F:3-4	260	95* + 15	150	0 in topping* 2mm in original grout topping	40-45			
C7	Near Grids W-V:4-5	250	100* + 5	145-150	0	45-65			
C8	Near Grids U:3-4	240	90* + 10-12		0 in topping* All of original grout and 2-5mm in original slab	35-40	18	15	
C9	Grids M:7	250	105* + 17	145	0 in topping* 10-12mm in original grout topping	40			

- The * denotes topping slab likely added post 2000.

APPENDIX J
JOINT REPORT: BEAM TESTING

COVEY
ASSOCIATES PTY LTD
Consulting Engineers

JOINT REPORT - CARBONATION TESTING ORIGINAL CONCRETE BEAMS CB1 & CB2

AT

PAVILION 1, 50 MARINE PARADE, REDCLIFFE

BY

ACOR CONSULTANTS & COVEY ASSOCIATES

FOR

MORETON BAY REGIONAL COUNCIL

PROJECT NO: 223164
REF: RG/RG/27736RPT – A
DATE: 22 DECEMBER 2022

DOCUMENT ISSUE APPROVAL

Project No:	223164
Title:	Carbonation Testing – Original Concrete Beam – Pavilion 1
Client:	Moreton Bay Regional Council
Date:	22 December 2022
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CONTENTS

1.0 BACKGROUND

The original concrete beam is assumed to have been poured in-situ with the suspended concrete slab in Pavilion 1. The section of beam related to this report is labelled 4:C/W. The beam has evidence of previous repair patches in many places – see photos and locations in Appendix A. BG & E Engineers tested the beam in August at

- 9 locations for strength,
- 1 location for chlorides and
- 1 location for Sulfate

It was deemed necessary that carbonation testing was also required to validate the residual life assumptions.

2.0 METHODOLOGY

On 10 November 2022, breakouts were carried out along the original concrete beam along grid 4:C/W. See drawing in Appendix B for reference. The breakouts were done using a jack hammer. The breakouts were immediately sprayed with phenolphthalein to test for carbonation. The testing was carried out by the senior lead engineer from ACOR in the presence of a senior structural engineer from Covey. Test locations were spread out along the beam.

The location of the breakouts are marked on the plan in Appendix B and the summary of the results is tabulated in Table 1 in section 4 below.

Three breakout areas chosen by ACOR to cover the following:

1. An existing repair area (BR1)
2. An area with existing defects (BR2)
3. An area that appeared to be in good condition with no existing defects (BR3)

ACOR stopped testing after these first 3 breakouts all produced similar results, hence there was no need to continue testing the rest of the beam. See section 4.

3.0 EXISTING REPAIRS & DEFECTS

The main beam along grid 4 has many existing repairs and defects as indicated by the plan and corresponding photos in appendix A.

4.0 TEST RESULTS

Breakouts of the concrete area labelled BR1 to BR3. It was found that there were some vertical ligs/hook bars with15 to 20mm side cover and 20-35 bottom cover. BG & E engineers scanned the beam and identified 3 bottom bars. The breakouts revealed the cover to these bottom bars ranged between 33 and 40mm. The carbonation zones extended to at least 40mm from the side and soffit, hence all reo including ligatures/hook bars and bottom bars are in carbonated concrete.

Table 1

No.	Lig found	Side Cover to Lig (mm)	Soffit Cover to Lig (mm)	BTM Bar Found	Side Cover to BTM Bar (mm)	BTM Bar cover from Soffit (mm)	Depth of Carbonation from the side (mm)	Depth of Carbonation From Beam Soffit (mm)	Notes
BR1	YES	20	20	YES	35	33	40	40	Reinforcement
BR2	YES	15	35	YES	N/A	N/A	35-40	35-40	BTM Bar lower than lig here
BR3	NO	N/A	N/A	YES	?	?	40	40	Reo has minor surface rust

BR1

This breakout was undertaken in an area where there was previous repairs and patching in order to determine how well the patch repairs were done. Note the before and after images below. The breakout continued past the patched area to a section of beam where there was no patch.

Image 1: Before Breakout BR1

Image 2: After Breakout BR1

Clearly the reinforcement is in the carbonated zone and is rusting, expanding and delaminating where the previous repair patch was installed as well as in the extended breakout area.

BR2

This breakout was undertaken in area where there was an existing defect. See before and after images below.

Image 3: Before Breakout BR2

Image 4: Concrete delaminated easily at BR2

Image 5: Breakout at BR2 exposed bottom bar and lig – phenolphthalein spray still visible

Image 6: Further breakout at BR2 exposed bottom lig – phenolphthalein sprayed at lig

BR3

This breakout was carried out in the centre area of the beam that would have been the most protected over the years and where there appeared to be no defects.

Image 7: Before Breakout BR3

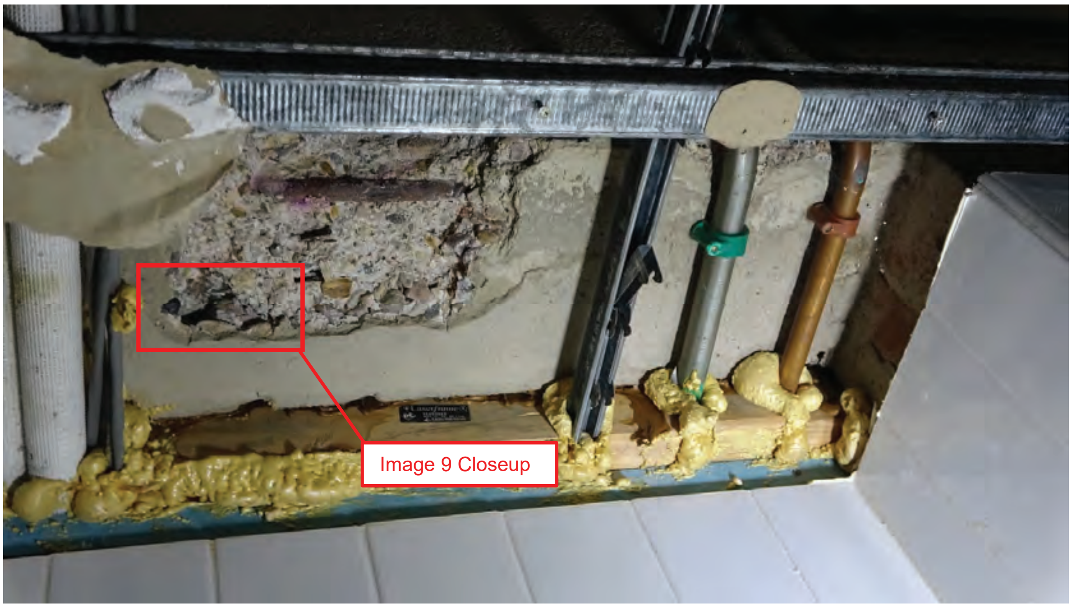

Image 8: Breakout BR3 – Fully Carbonated past bottom bar

Image 9: Close up from image 8 shows

The reinforcement at this location appears to be in reasonable condition with minor surface rust, no loss of section or delamination.

5.0 DISCUSSION / CONCLUSIONS

The carbonation depth exceeds the depth of reinforcement in the soffit and side of the beam CB2. This indicates that the protective passive layer provided by the concrete to the bottom reinforcement and ligatures is already gone and at some locations the steel has started corroding. Cracking and spalling of concrete has also begun in some areas.

Existing repair areas are compromised as reinforcement beneath repairs is corroding so it is only a matter of time before the steel reinforcement begins to delaminate, followed by cracking and spalling of the concrete in these areas. See report in Appendix Q of Main Report.

6.0 LIMITATIONS

The opinions, conclusions and any recommendations in this report are based on information from, and testing undertaken at or in connection with, specific sample points. Site conditions in other parts of the site may be different to those found at the specific sample points.

The opinions, conclusions and recommendations in this report are based on the assumptions made by Covey Associates and ACOR described in this report. Covey Associates and ACOR disclaim any liability arising from any of these assumptions being incorrect.

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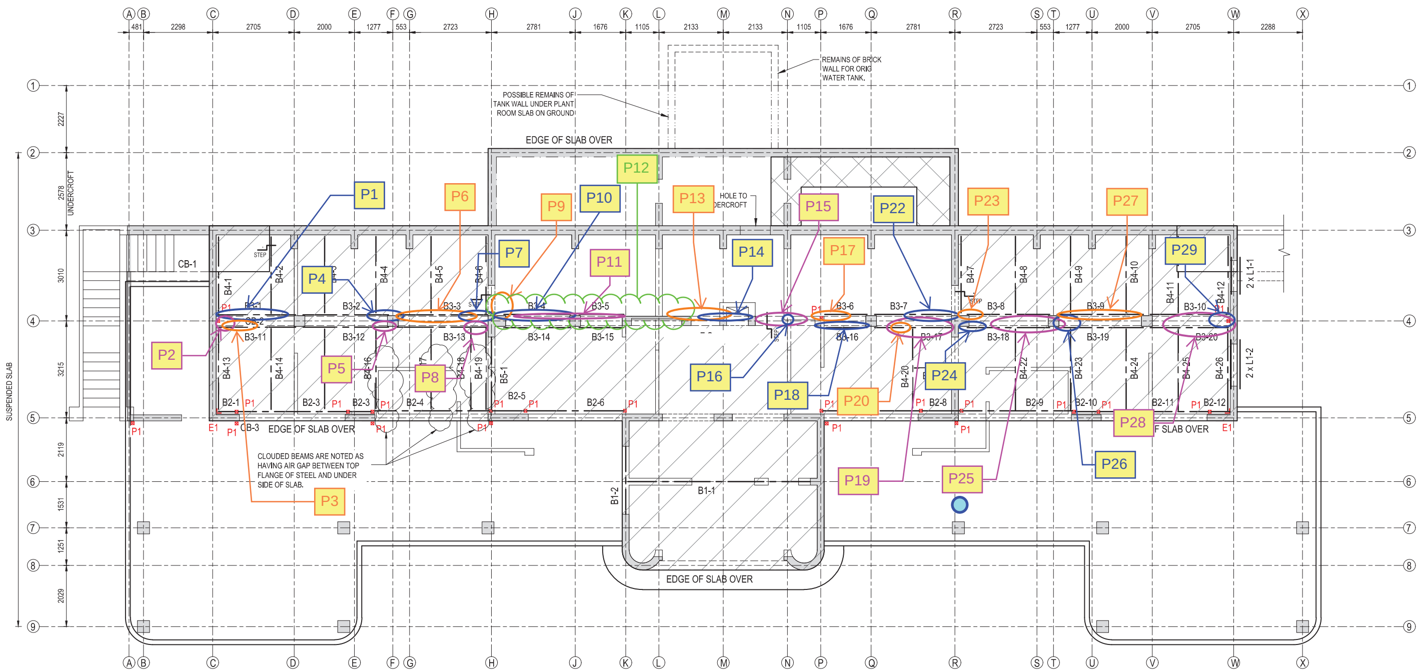
APPENDIX A

EXISTING REPAIRS AND DEFECTS

LOCATION PLAN AND CORRESPONDING PHOTOS



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ID	DESCRIPTION	MATERIAL	ORIGIN
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EA1	150x10 EA	STEEL	POST 2000
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B3	200 PFC	STEEL	POST 2000
B4	180 PFC	STEEL	POST 2000
B5	150PFC	STEEL	POST 2000
L1	100EA x 6	STEEL	POST 2000
CB-1	GRID A - C	CONCRETE	ORIGINAL
CB-2	GRID C - W	CONCRETE	ORIGINAL
CB-3	GRID C - W	CONCRETE	

NOTE:

ALL DIMENSION ARE BASED OFF SCANNED PDF FROM 1937 ARCHITECTURAL SKETCHES, ROUGH SITE MEASURES. DIMENSIONS ARE APPROXIMATELY +/- 100mm
FOR MORE ACCURATE PLANS SEE DRAWINGS PRODUCED BY 3D CLOUD POINT (BY OTHERS).

REFERENCE PLAN - SHOWING LOCATIONS OF PHOTOS OF PREVIOUS REPAIRS AND DEFECTS

LEGEND

- ORIGINAL 1937 ARCHITECTURAL LAYOUT
- CURRENT LAYOUT NOM. 100 mm TOLERANCE
- BEAMS OVER
- BEAMS UNDER
- UNDERCROFT WALLS
- CURRENT LOWER SUPPORTING WALLS / COL.

- HATCHED AREA INDICATES TOPPING SLAB OVER ORIGINAL SLAB
- 100mm HIGH CONCRETE PLINTH OVER ORIGINAL SLAB
- STEP — ORIGINAL STEP PRIOR TO TOPPING

Check all dimensions before commencement of work. Check Site boundary dimensions from the Title plans Check Building Boundary clearances by set out. Notwithstanding the information supplied on this drawing, the location, depth and extent of underground or overhead services are to be confirmed and protected on site by the contractor prior to commencement of works.

Checked – RG
Design – CJT Drawn – MS
Scales – AS SHOWN
Document Stage – INFORMATION

Issue	Date	Description	By
E	02.11.22	ISSUED FOR INFORMATION	TW
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B	20.07.22	ISSUED FOR INFORMATION	CJT
A	15.07.2022	ISSUED FOR INFORMATION	MS

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Drawing title – PAVILLION 1 UPPER FLOOR FRAMING STRUCTURE

Project – SUTTONS BEACH PAVILIONS

Client – MORETON BAY REGIONAL COUNCIL

Site – SUTTONS BEACH, REDCLIFFE, QLD, 4021

Project No. 223164 Sheet No. SK102
Digital Ref : 223164_SK102 FLOOR FRAMING STRUCTURE Issue : E

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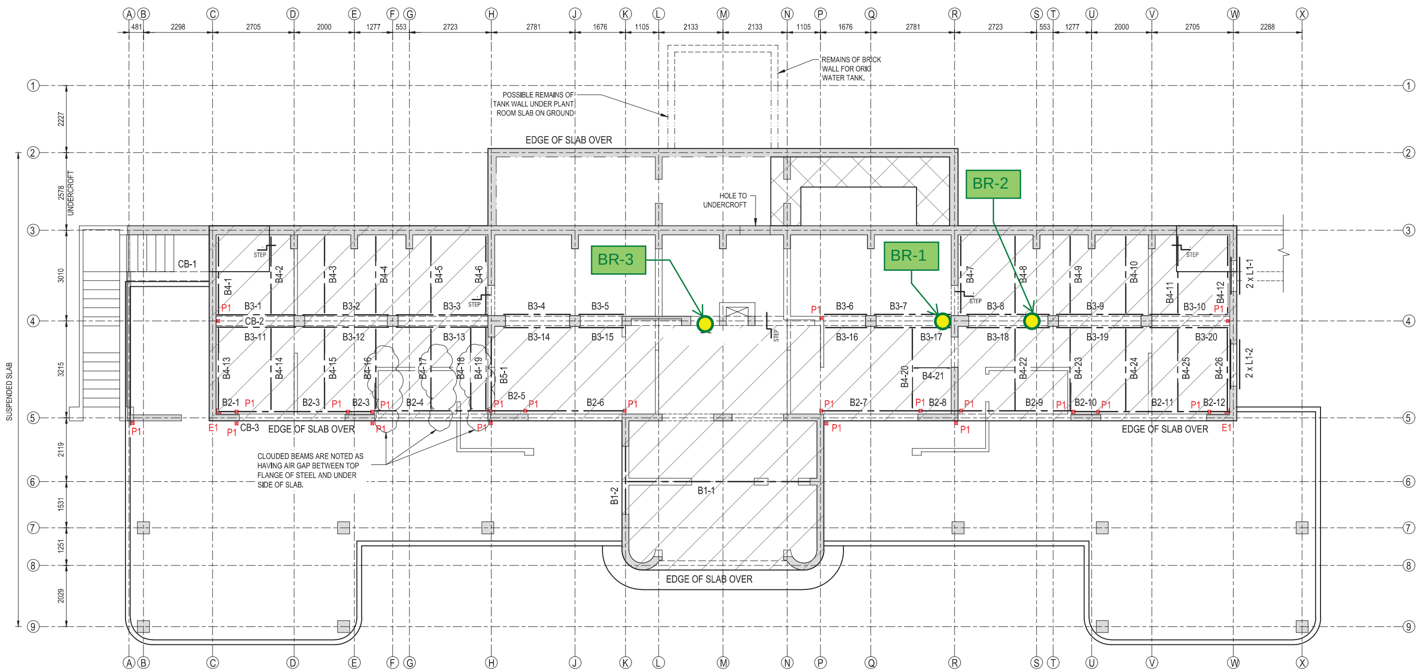
Plot 04 Nov, 2022 - 09:25

APPENDIX B

CARBONATION TEST LOCATIONS
FOR
CONCRETE BEAM ON GRID 4:C/W



LOCATIONS OF CARBONATION TESTS AND BREAKOUTS ALONG ORIGINAL CONCRETE BEAM

STRUCTURAL ELEMENT - IDENTIFICATION

ID	DESCRIPTION	MATERIAL	ORIGIN
P1	100 SHS	STEEL	POST 2000
EA1	150x10 EA	STEEL	POST 2000
B1	360UB 50.7	STEEL	POST 2000
B2	250PFC	STEEL	POST 2000
B3	200 PFC	STEEL	POST 2000
B4	180 PFC	STEEL	POST 2000
B5	150PFC	STEEL	POST 2000
L1	100EA x 6	STEEL	POST 2000
CB-1	GRID A - C	CONCRETE	ORIGINAL
CB-2	GRID C - W	CONCRETE	ORIGINAL
CB-3	GRID C - W	CONCRETE	

NOTE:

ALL DIMENSION ARE BASED OFF SCANNED PDF FROM 1937 ARCHITECTURAL SKETCHES, ROUGH SITE MEASURES. DIMENSIONS ARE APPROXIMATELY +/- 100mm
FOR MORE ACCURATE PLANS SEE DRAWINGS PRODUCED BY 3D CLOUD POINT (BY OTHERS).

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LEGEND

- ORIGINAL 1937 ARCHITECTURAL LAYOUT
- CURRENT LAYOUT NOM. 100 mm TOLERANCE
- BEAMS OVER
- BEAMS UNDER
- UNDERCROFT WALLS
- CURRENT LOWER SUPPORTING WALLS / COL.

- HATCHED AREA INDICATES TOPPING SLAB OVER ORIGINAL SLAB
- 100mm HIGH CONCRETE PLINTH OVER ORIGINAL SLAB
- STEP — ORIGINAL STEP PRIOR TO TOPPING

Check all dimensions before commencement of work. Check Site boundary dimensions from the Title plans Check Building Boundary clearances by set out. Notwithstanding the information supplied on this drawing, the location, depth and extent of underground or overhead services are to be confirmed and protected on site by the contractor prior to commencement of works.

Checked – RG
Design – CJT Drawn – MS
Scales – AS SHOWN
Document Stage – INFORMATION

Issue	Date	Description	By
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Approved

Drawing title – PAVILLION 1 UPPER FLOOR FRAMING STRUCTURE

Project – SUTTONS BEACH PAVILIONS

Client – MORETON BAY REGIONAL COUNCIL

Site – SUTTONS BEACH, REDCLIFFE, QLD, 4021

Project No. 223164 Sheet No. SK102
Digital Ref : 223164_SK102 FLOOR FRAMING STRUCTURE Issue : E

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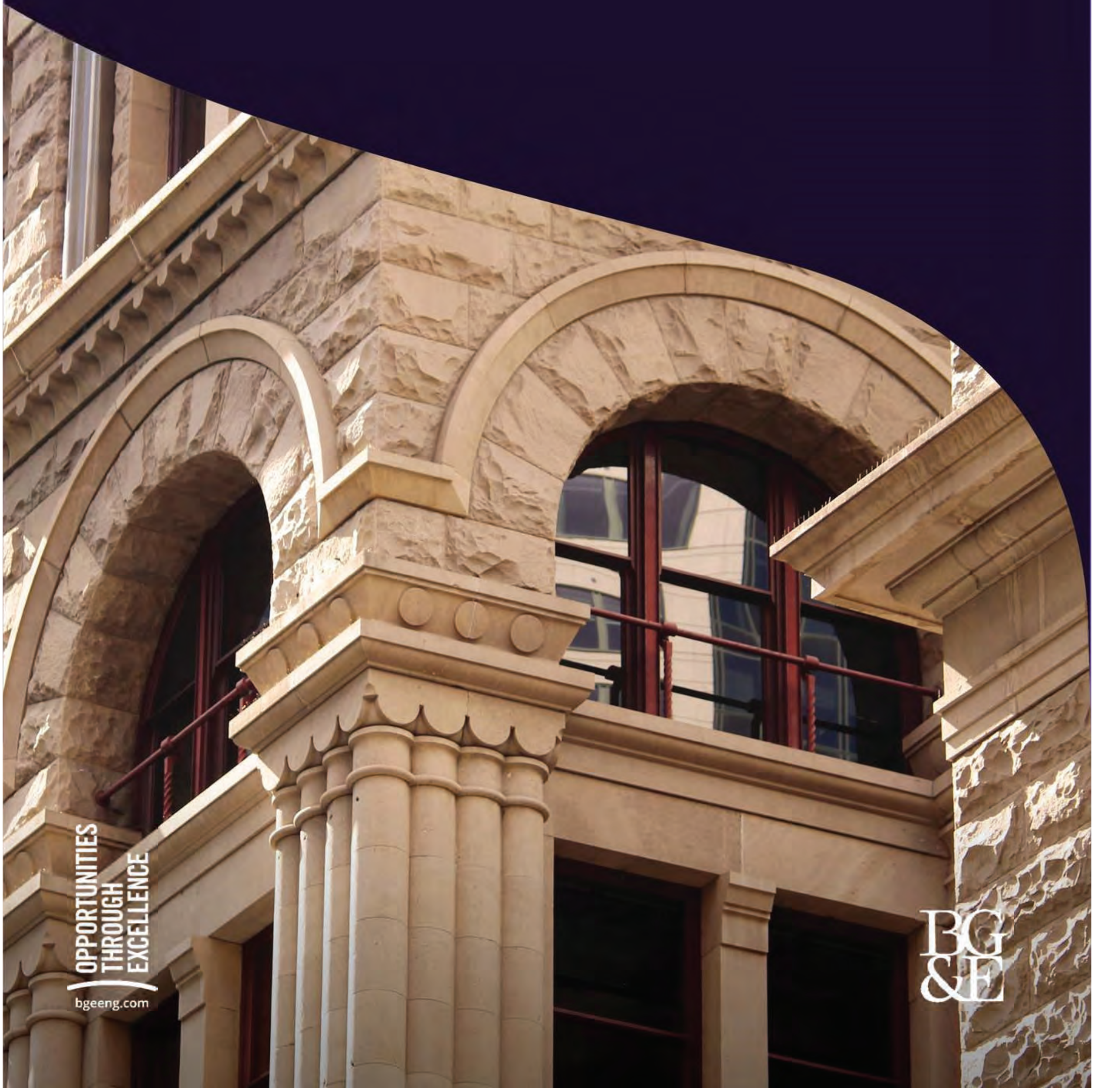
Plot 04 Nov, 2022 - 09:25

APPENDIX K
STRUCTURAL ASSESSMENT OF THE REDCLIFFE PAVILION SUSPENDED SLAB BY BG&E

Sutton's Beach Pavilion Structural Assessment Report

Prepared for Covey Associates

November 2022
Project Number B20205



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Document Control				
Revision	Date	Prepared	Reviewed	Approved
A	24/11/22	HM	BDS	BDS
B	02/12/22	HM	BDS	BDS

1. Executive Summary

BG&E Pty Ltd were engaged by Covey Associates Pty Ltd to complete a Structural assessment of the existing suspended slab at Suttons' Beach Pavilion, located on 50 Marine Parade, Redcliffe QLD.

BG&E have previously undertaken a condition assessment of the slab, Sutton's Beach Pavilion Condition Assessment Report, Rev C, 21/11/22, By BG&E, which confirms the slab is in poor condition and beyond its design life for durability.

This structural assessment has found the existing slab is not fit for purpose, is not capable of carrying any additional loads and is potentially susceptible to brittle failure.

The two options for ensuring the slab is fit for purpose are:

1. Floor replacement.
2. Major rectification.

Floor replacement is reasonably standard, and a scope of works could be confirmed with a lump sum fee agreed with a Contractor to undertake the works.

A specific scope of works for the floor rectification cannot be confirmed as it will be an iterative process involving the Engineer and Contractor on site to determine slab suitability for retention during the demolition process. It is also likely that the costs to undertake rectification would significantly exceed the cost of slab replacement.

2. Introduction

2.1 Background

Sutton's Beach Pavilion, located in Redcliffe, was constructed in 1937 and has since undergone numerous renovations. A structural and condition inspection of the Redcliffe Pavilion, previously undertaken by BG&E Materials identified some corrosion of reinforcement, raising concerns with the suspended slab and western concrete retaining wall. A detailed condition assessment was required to determine the anticipated residual life of these elements and the structural capacity of these elements should they be repairable.

2.2 Scope of Works

Following the previously released Condition Assessment Report (Sutton's Beach Pavilion Condition Assessment Report, Rev C, 21/11/22, By BG&E), this report highlights the findings from BG&E's structural assessment of the concrete suspended slabs. The assessment takes into consideration the structural load carrying capacity of the original design and the condition of the slabs based on the finding of the afore mentioned BG&E report.

For the purpose of this report the suspended slab of the Redcliffe Pavilion is broken into several areas based on the condition and any additional rectification measures that have been taken out on the slab. These are highlighted in the below figure.

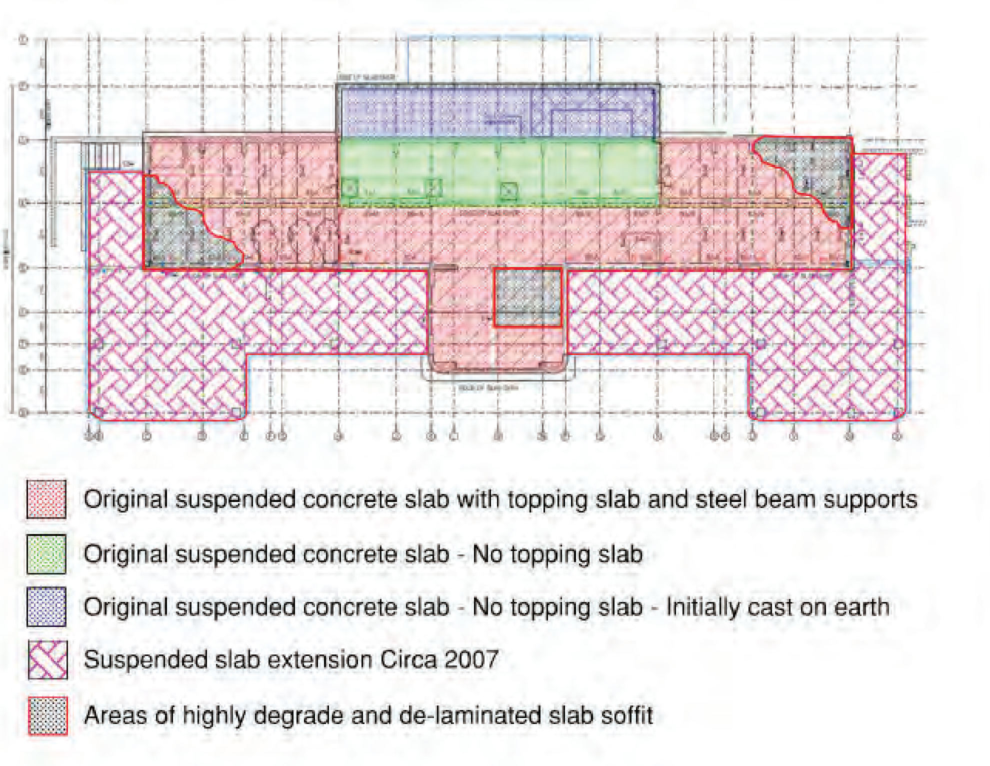

Figure 1 – Slab Identification Plan

This report focuses on the area highlight in RED noted as "Original suspended concrete slab with topping slab and steel beams supports" and the areas in GREEN and BLUE, noted as Original suspended concrete slab'.

This report does not consider the condition or structural integrity of the suspended slab extension highlighted in pink on the above image.

3. Structural Assessment of the Suspended Slab with Topping Slab (Red and Black in Figure 1)

3.1 Original Suspended Slab Assessment

The original suspended slab is reinforced cast in-situ concrete with a topping layer over the entire floor. Additionally, there is structural steel frame that supports the suspended slab.

The original suspended slab has a single layer of reinforcing at the bottom of the slab, consisting of ¼ inch (6.3 mm) bottom layer bars at 4-inch (100 mm) spacing in the transverse (Grid direction A-X) direction and ¼ inch (6.3 mm) bars at 12-to-16-inches (300 to 400 mm) spacing in the longitudinal (Grid direction 1-5) direction, with 20 to 40 mm of cover to the soffit.

The topping layer did not contain any steel reinforcement and no ties were found at the interface between the back wall and the slab. No top steel or shear reinforcement was identified at the tops of the columns.

3.2 Slab Structural Assessment

BG&E have undertaken a structural assessment of the original slab and concrete beam spanning along grid as per the above findings. BG&E have assumed the slab is in pristine condition for this assessment to confirm the potential design loading criteria of the slab under existing design code requirements (AS3600-2018).

The assumptions for design are:

1. Slab is 150 thick spanning a two 2.8m spans (one way spanning slab)
    a. This design check was undertake on the two span, one way slab along grid D, spanning between grids 3 to 5.
    b. A second slab design check was done on the original slab spanning along grid 3.5 between grids C to H using the steel beams as supports.
2. 6.3mm bars at 100mm spacing is equivalent to 311mm^2 of low grade (250MPa) steel in the longitudinal span direction in the bottom of the slab per metre width.
3. 6.3mm bars at 300 - 400mm spacing is equivalent to approx. 100mm^2 of low grade (250MPa) steel in the latitude span direction in the bottom of the slab per metre width.
    a. This would be equivalent to approximately 160mm^2 of 500MPa steel
4. The strength of the concrete beam that runs along grid 4 cannot be confirmed as no information is available regarding the reinforcement in the beam.

BG&E findings for the capacity of the original slab, without the topping slab or steel beams, is as follows:

1. The original slab **does not** have sufficient reinforcement to support its own self weight in the longitudinal slab span direction (as originally designed), according to current design standards.
2. The original slab **does not** have sufficient reinforcement to support its own self weight in the latitude slab span direction using the steel beams as support.
3. The original slab **does not** have the capacity to support it's own self weight as a two way spanning slab using the longitudinal 2 way span and the latitude span direction with the steel beams as supports.
4. The original slab **does not** have sufficient capacity to support any additional dead or live load, according to current design standards.

The condition of the original slab is also considered to be poor and beyond its design life for durability, as outlined in the Condition Assessment Report.

3.3 Steel Beam Assessment

3.3.1 Methodology

A structural assessment of the existing steel beams was performed to determine the capacity of the system. Structural analysis was performed using a combination of first principle static analysis and computer modelling with the computer analysis program Space Gass. Capacities of the steel members were determined using AS4100 (2020).

3.3.2 Assumptions

A summary of the loading assumptions for this assessment is located below. As the original suspended slab is considered redundant in the design, both it and the topping slab are considered as extra dead load with a maximum thickness of 150mm plus 260mm. Load combination cases are in accordance with AS1170.1.

Table 1 -Structural Loading Assumptions

Load Type	Action
Concrete Unit Weight	25 kN/m^3
Steel Unit Weight	7850 kN/m^3
Original Concrete Slab and Topping slab (150mm + 260mm)	10.25kN/m^2
Super Imposed Dead Load	1.5 kPa
Live Load	4 kPa

For simplicity of analysis, the frame system is treated as an arrangement of simply support secondary beams (150 PFC), bearing onto primary beams (200PFC) as point loads which span between columns and posts (See Figure 1).

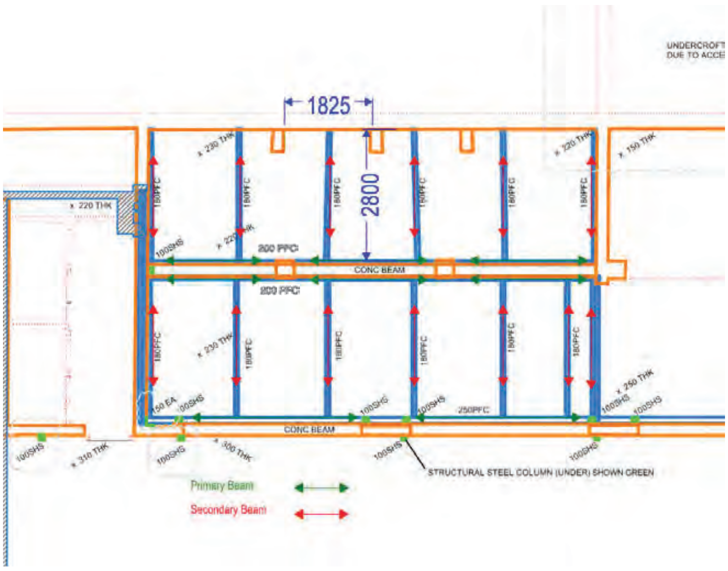

Figure 2 – Type A Structural Framing Layout

There is also a 360 UB 51 spanning along grid 6 between grids K and P. This is assumed to have no top chord fixity and spans approximately 6.3m. This beam has been assessed based on the current dead load of the concrete slab and a live load of 4kPa. Results are displayed in Table 2.

This beam does not have capacity to support the slab weight plus an additional live load of 2kPa.

3.3.3 Results

Based on the analysis, the steel beams have sufficient capacity to support the above loads including the concrete slabs and additional live loads. The maximum strength and deflection utilisations are summarised below.

Table 2 – Steel Beam - Strength Assessment

Beam	Capacity (kNm)	Action (kNm)	Utilisation
PFC180	34	28.1	83%
PFC200	50	32	64%
PFC250	54.9	54.9	70%
360 UB 51	93	130	141%

Table 3 – Steel Beam – Deflection Assessment

Beam	Max Deflection (mm)	Deflection Limit Span/250	Utilisation
PFC180	5.5	11.2	49%
PFC200	4.7	12.6	37%
PFC250	6	15.8	38%
360 UB 51 (assessed with 2kPa Live Load)	10.36	25.2	41%

3.3.4 Overall Floor System

The current floor system appears to follow the below structural methodology:

1. Steel PFC beams supporting original slab
2. Original slab supporting additional concrete topping slab

The structural assessment of the system confirms that the steel beams have sufficient capacity to support the concrete slabs, but that the concrete slabs do not have sufficient capacity to support any load. This is based on the following factors:

1. Original slab does not have sufficient design capacity to support additional loads, as assessed against current design standards (AS3600-2018).
2. Topping slab does not have sufficient design capacity to support additional loads, as assessed against current design standards (AS3600-2018).
3. The condition of the original slab is poor and includes severe corrosion of reinforcement and concrete spalling.

3.3.5 Effect of Corrosion on reinforced slabs

As noted above, the original slab shows signs of significant corrosion to its reinforcement. Reinforcement is used in concrete suspended slabs to allow the slabs to work in a ductile manner. That is, as the slabs are loaded, they deflect downwards, and the reinforcement stretches in a ductile manner until the slab is overloaded. The reinforcement then begins to permanently deform causing further, excessive slab deflection, before eventual failure and collapse. This process takes time and is predictable. It would generally allow the slab to be unloaded prior to collapse.

If the reinforcement is not present (or in this case, severely corroded), the slab has no ability to deflect slowly prior to collapse. This is called brittle failure and can occur in an instant. This failure is also not predicable. It is BG&E's assessment that the condition of this slab has deteriorated to the point that brittle failure may be a possibility.

4. Structural Assessment of the Original Suspended Slab Areas (Blue and Green in Figure 1)

This section of slab is similar in construction to the original 150mm thick slab noted in Part 1 of this assessment. The blue section of slab (figure 1) appears to have been constructed as a suspended concrete slab, cast on ground with the supporting earth later excavated. The green section of slab appears to have been cast on more traditional formwork.

No further calculations have been undertaken to confirm the capacity of this slab. However; by inspection this slab capacity will be similar to the slab described in section 3.2 of this report as it has similar depth and spans. This section of slab is not compliant with the current AS3600 Concrete Code, or AS1170.1 SSA Loading Code.

This section of slab does not have any steel beams, nor additional topping slab.

The condition of this portion of slab is not as poor as the section discussed in section 3.2. However; as noted in BG&E's previous condition report, the reinforcement in this section of slab is electrically linked to the reinforcement throughout the slab. This means it will likely have undergone some corrosion or be at risk of corrosion.

This portion of slab has also exceeded its design life further exposing any reinforcement to potential corrosion and should be treated in a similar manner to part 1 of the slab for any rectification or re-instatement.

5. Recommendations and Conclusion

Due to the carbonation, spalling and visible steel reinforcement corrosion of the slab referenced in the condition assessment report (B20502-RPT-001-B), the original suspended slab is considered not fit for purpose and beyond its design life. It is the opinion of the structural engineer, that the slab has no residual structural capacity and does not comply with AS3600 (2018).

The existing steel framing under the original slab is suitable to support a dead load equivalent to a 260 thick slab, super imposed dead load of 1.5 kPa and a live load of 4 kPa. The framing could be reused and incorporated into a replacement structural system and should be kept in place during any demolition of the original slab.

As noted in the previously mention Condition Report, the two options for ensuring the slab is made fit for purpose are:

1. Floor replacement
2. Major rectification.

The major rectification works would likely constitute the following requirements as a minimum:

1. Removal of any damaged slab or carbonated concrete
2. Chase all corroded reinforcement in the slab until clean, non-corroded reinforcement is confirmed and treat the exposed reinforcement with a rust inhibitor / epoxy coating.
3. Reinstate all reinforcement with new reinforcement.
    a. It should be noted that this will be required on the top surface of the slab and on the soffit in order to make the slab compliant to modern design standards.
4. Reinstate all removed concrete with a repair mortar or new concrete.
    a. This would likely require the entire bottom and top surfaces of the slab to be re-instated.

It is the opinion of the author that attempting to repair and reinstate the slab would likely result in the almost complete removal of the slab due to the extent of concrete carbonation and corrosion of reinforcement. The costs to undertake this type of rectification would also almost certainly exceed the cost of removal and replacement of the slab by a significant factor.

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Appendix L
Footings Report

COVEY
ASSOCIATES PTY LTD
Consulting Engineers

FOOTINGS

AT

PAVILION 1, 50 MARINE PARADE, REDCLIFFE

FOR

MORETON BAY REGIONAL COUNCIL

PROJECT NO: 223164
REF: RG/RG/27751RPT – A
22 DECEMBER 2022

DOCUMENT ISSUE APPROVAL

Project No:	223164
Title:	FOOTINGS
Client:	Moreton Bay Regional Council
Date:	December 2022
Issue No:	Draft - D1
Distribution:	Moreton Bay Regional Council – One (1) copy Covey Associates – One (1) copy

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CONTENTS

1.0 OBSERVATION

1.1 Component Description

Original concrete cast-in-situ strip and pad footings and sub floor walls.

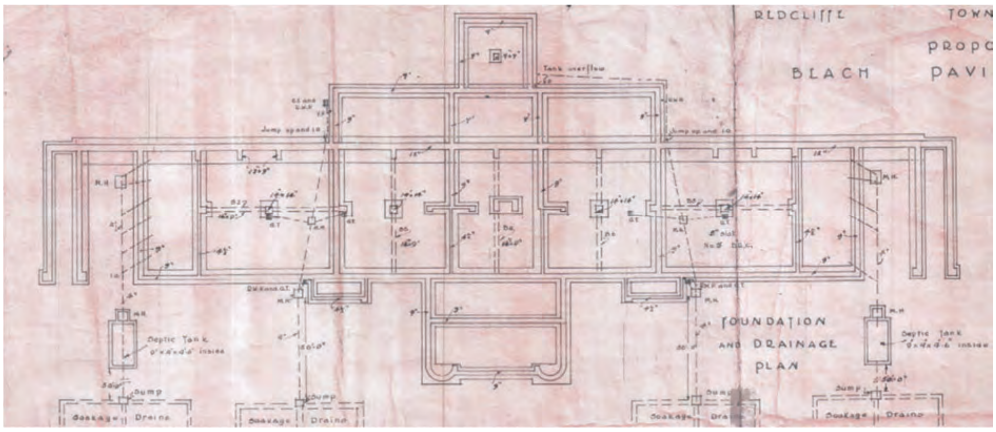

Figure 1: Excerpt from original blue prints showing footings and sub floor walls.

1.2 Component Reference

Grid line references are used to describe footings at grid locations.

1.3 Construction

- Mass concrete strip footings.
- Stepped at grids 2 and 3, 3 and 4.
- Short sub floor brick walls built above concrete footings.
- Sandy clay fill between subfloor walls.
- Slab on Ground poured over.

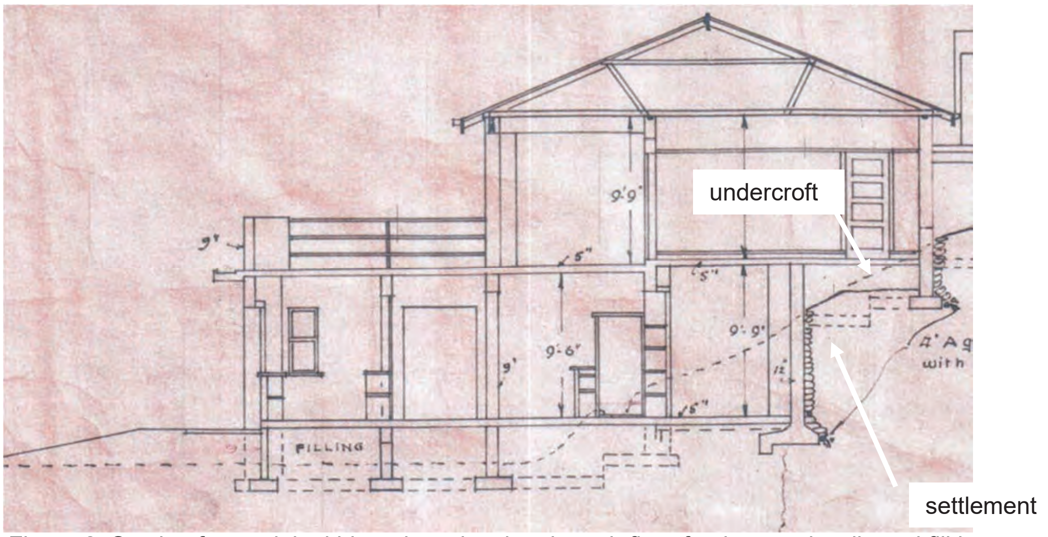

Figure 2: Section from original blueprints showing the sub floor footings and walls and fill between.

1.4 Structural Components and details

- The footings exposed during excavation were generally built, in accordance with the original blueprint. Although there was no front toe/key on the rear retaining wall footing found.

1.5 Reinforcement

- All reinforcement scanning was carried out by others.
- Scans by others revealed no reinforcement in the original concrete footings.

1.6 Geotechnical Parameters

- 3 footings on the existing building were exposed and test pits assessed by a geotechnical engineer including:
  - Rear retaining wall footing internally at 3-R/S
  - Brick wall footing at 7:N/P
  - Brick column footing at 4:D
- Externally a section of the retaining wall was exposed at approximately 3-C/E. The depth of excavation resulted in benching being required and a number constraints (physical, electrical, geotechnical and safety) meant only the top of the rear external footing could be exposed. See retaining wall section for more details.
- A geotechnical engineer carried out tests and was present for footing excavations including the rear retaining wall at 3-D/E. See results in appendix G of main report.
- The Geotech engineer stipulated good foundation material with a moderate site reactivity classification (Class M).
- The geotechnical investigation also revealed that the clay fill was firm to stiff with allowable bearing pressures of 120kPa to 150kPa.

1.7 History

The area of slab shown highlighted looks like it was always supposed to be a suspended slab above an undercroft. Observations show the soffit was poured on the ground and was later dug out. It has a perimeter wall and 2 intermediate walls along grid lines L and N between grids 2 and 3. See observations section for more information.

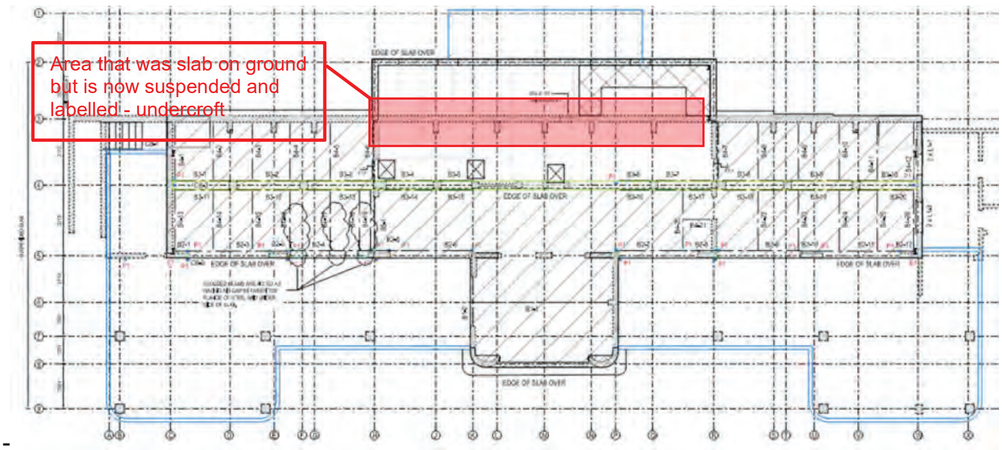

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Figure 3: Upper floor framing plan highlighting the undercroft area.

2.0 OBSERVATIONS

Detailed observations, comments with supporting photos. Footing exposure locations confirmed with heritage and geotechnical engineer. All footings were excavated using vacuum truck which was the preferred method by heritage consultant. All footings excavated were generally in accordance with the original blueprint footing plan. Generally, footings and sub floor walls appeared to be in good condition.

There were no major signs of distress or movement in the new section of the front of the building. A few areas in the original building and rear plant room had cracking – typically associated with foundation and footing movement. See itemised areas below.

2.1 Footing at 8-N/P

The external slab in front of the brick wall was cut and excavated using vacuum truck in the presence of the heritage consultant and the geotechnical engineer. It was found that the brick wall beneath the ground floor slab was approximately 700mm deep. The bricks and mortar appeared to be in excellent condition. The sub floor wall was sitting on a concrete footing approximately 300 deep.

Photo 1: Footing exposure at 8:N/P.

2.2 Footing at 4-D

The internal slab was cored and cut at this location. Again, there was approximately 750mm of bricks between the concrete footing and slab on ground. The sub floor brick wall was again supported on a concrete strip footing which stepped up towards grid 3. There was a concrete wall along grid 4 as per the plan. The original slab on ground was approximately 150mm thick with reinforcement in the bottom only with little to no cover. There was newer concrete above this - a 70mm topping slab with no reinforcement. A section of the slab was thicker and newer with reinforcement spanning the PVC pipe (edge exposed during the vacuum excavation).

Cracking stepping up in the mortar between the bricks was noticed – this indicates movement at some stage. Possibly due to consolidation of fill and/or settlement of the back corner at 3:C-D. See section in figure 4 below.

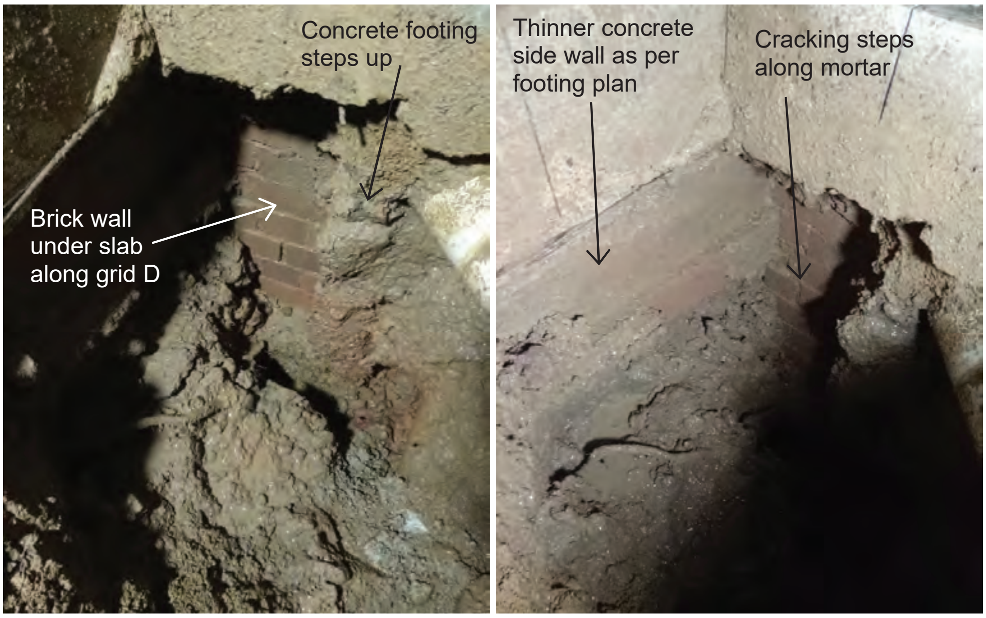

Photo 2: Sub floor wall and footing supporting Column at grid D:4.

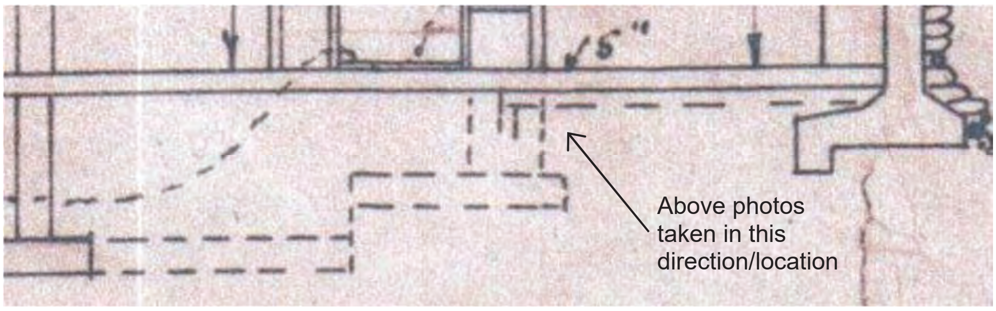

Figure 4: Original building section showing footings.

Photo 3: Underside of original slab on ground. Note reinforcement with no cover and some corrosion.

2.3 Footing at 9-X

The external slab was cut and vacuum excavated to expose the concrete footing under the new column from the 2007 extension. The external slab was approximately 250mm deep over a concrete footing that was 450mm deep. A random exposed reinforcement bar was found in contact with the dirt with visible corrosion and sectional loss of steel.

Photo 4: Vacuum excavation at 9-X. Note stormwater pipe runs vertically inside the concrete column and then out here above the footing.

Photo 5: Exposed reinforcement bar and concrete footing at 9-X (built in 2007).

2.4 Retaining wall footing – internally at 3-R/S

The internal slab and topping slab were cut out and vacuum excavated to expose the toe of the retaining wall. No key was found like shown on the original blueprints. Dimensions as per geotechnical references and sketch shown below.

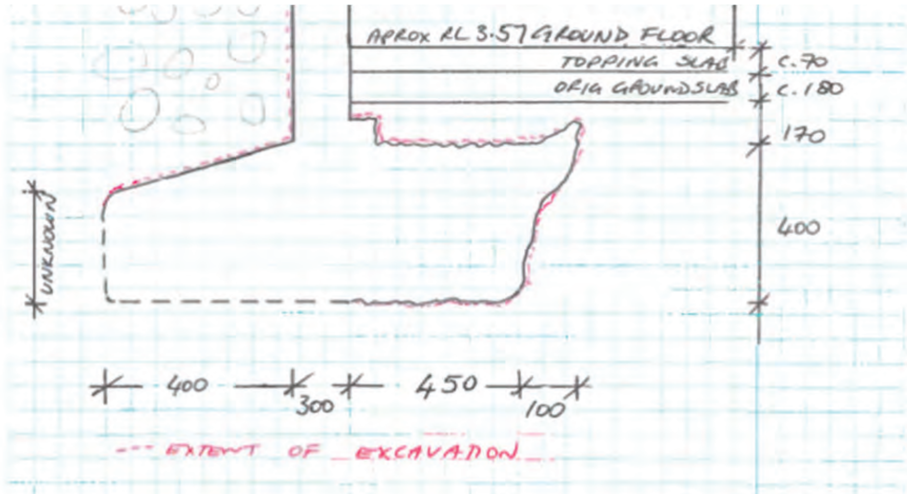

Figure 5: Retaining wall footing excavation – measured internally at 3-R/S and externally at 3-C/E.

Photo 6: Vac Excavation showing footing at 3-R/S. Also note topping slab and slab on ground under.

2.5 Retaining wall footing – externally at 3-C/E

Externally the rear retaining wall was vacuum excavated to the top of the footing. This was due to geotechnical, site and safety constraints. The external measurements taken are shown in the sketch detail in the previous section.

Photo 7: Excavation of rear retaining wall at 3:C-E.

2.6 High level footings in undercroft at N:2-3 and 2:M-N

During the strip out a hole to the undercroft was found. Through it the suspended slab could be seen. Note this section of the suspended slab was originally slab on ground as discussed in the history section above.

Photo 8: Hole in rear retaining wall at 3:M-N.

Photo 9: Taken through hole in rear retaining wall showing undercroft area. Note uneven soffit to suspended slab and same colour dirt – indicating that the slab was originally poured on ground. Also note the mass concrete footing200-300 deep stepping up towards the back near grid 2. The brick wall was much wider than double skin. It appears to be triple skin brick with a number of removed and broken out bricks to install sewage
pipes etc.

Photo 10: Diagonal cracking and gaps in undercroft sub floor support wall. Footing appears to be at least 300mm thick. Cracking indicates movement – the subfloor brick wall and footing has settled behind the rear retaining wall. This footing must not be tied to the rear retaining wall and can thus move/settle separately. See Geotech report for more details.

2.7 Footing movement at wall C-3/4, 3-C/D and buttress nib 3-D

Note there is cracking in wall C-3/4, 3-C/D and buttress/nib at 3-D. See *Ground Floor Walls and Columns* for more details. These cracks suggest settlement of the rear retaining wall footing in this area. In addition to unknown/inadequate drainage behind the rear retaining wall, the cracked stormwater pipe could be providing a water source to flush fines away and consolidate foundation soils allowing this corner to settle. Causing the cracks and gaps visible today. See other sections of the report for more details including *Foundations* and/or *Leaks and Water Ingress* and/or Geotech report.

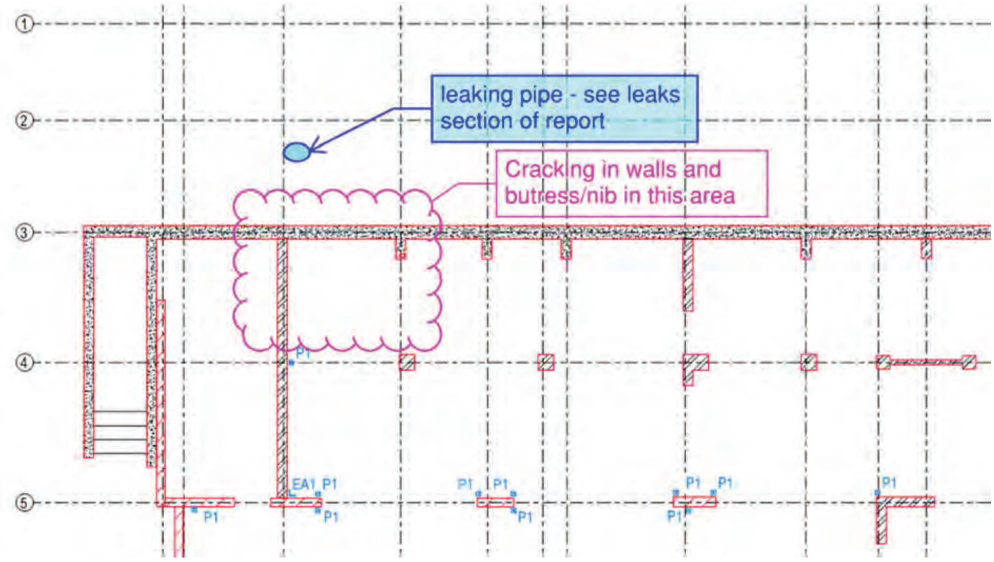

Figure 6: Plan showing corner with major cracking and leaking pipe proximity.

2.8 Other external cracking


Photo 11: External cracking along sub floor walls and footing supporting slab edge at G-2/3.

Photo 12: Stepped cracks in blockwork of plant room. Also note the horizontal crack near top left of window. These cracks indicate movement. Possibly settlement of high-level footings at the rear of the building along grid 1. Note the tree stump (termite ridden). Removal of this stump would have created a change in the moisture conditions of the surrounding soils, which in turn would affect the blockwork footings.

2.9 Summary of issues

Generally, footings and sub floor walls appeared to be in good condition. Although cracked areas indicate issues.

1. **Wall cracks:**
   In various locations indicate foundation issues or possible settlement of footings. In particular the Corner at 3:C.
2. **Wall cracks:**
   Settlement of sub floor brick wall footings behind rear retaining wall in undercroft.
3. **Ventilation:**
   There is no ventilation of sub floor brick walls at undercroft.
4. **Walls at undercroft:**
   Enclosed space. Cannot excavate behind wall or could undermine the higher-level footings along grid 2.

3.0 Discussion/Structural Assessment

For strength, serviceability, durability of footings both new and original. Note only a few areas were excavated so assumptions that the rest of the footings are in similar condition has been made.

3.1 Strength

In general, the foundation material the footings are founded in has an allowable bearing capacity of 120-150kPa and the induced loads from the structure are less than that hence the footings appear to be structurally adequate for current design loads. Footings appear to be mass concrete as no steel reinforcement was found in the excavated footings.

There is, however, no ties between the original footings, sub floor brick walls and slab on ground. The code requires these components to be tied together for lateral load transfer (e.g., earthquakes). It appears that in most areas the footings have been adequate over the last nearly 90 years. So, in general they can be considered to have adequate serviceability. There are however a few specific areas where there are structural cracks in the brick and concrete walls that indicate possible foundation movement or changes.

- At 3:C Ground Floor.
  The nature and positions of the cracks in brickwork, concrete wall, nib/buttress and suspended slab at 3:C is likely due to movement of footings.
- At W-4 Ground Floor.
  Large crack in brickwork is wider at the top than the base of the wall indicating it is a shear crack and structural failure of the bricks and perhaps not so much related to foundation movements.
- Undercroft (brick walls at N:
- At Upper rear brick wall along grid 2.
  Various cracking and shear cracking in brick walls, possibly due to brick….
  At Q:1-2 Upper Floor.
  Side wall of Plant room has cracking in block walls indicative of settlement.

At 9-X the depth of footing is typical for this type of modern construction. As there is no evidence of distress in the structure in the vicinity of the new footings or areas that are directly supported by new footings, they are thus deemed satisfactory for the current loading conditions.

3.2 Durability

BG&E concluded that the original and extension footings were considered to be in good condition, with no durability concerns.

3.3 Remedial options

In terms of durability there is no repair or maintenance for the footings required. See BG&E report.

Site water must be managed: all leaks must be fixed, all surface runoff and ground water must be drained appropriately away from the buildings foundations to minimise potential subsidence of loose sand and movement in the moderately reactive clays.

In terms of strength, the cracks and gaps in the sub floor brick walls in the undercroft should be filled (using approved masonry techniques) to provide adequate support of slab above.

It is recommended that the footings and supported elements at locations mentioned in section 3.1 above, be monitored at a minimum of 6 monthly intervals and if movement continues engage geotechnical and structural engineers to resolve.

Geotech recommendations include articulation of brickwork and providing adequate detailing and jointing to allow for some movement. Brittle finishes should be avoided.

APPENDIX M
JOINT REPORT – REAR RETAINING WALL

COVEY
ASSOCIATES PTY LTD
Consulting Engineers

Joint Report - Rear Retaining Wall

At

Pavilion 1, 50 Marine Parade, Redcliffe

By

Covey Associates Pty Ltd & ACOR Consultants

For

Moreton Bay Regional Council

Project No: 223164
Ref: RG/RG/37304Rpt – A
22 December 2022

Maroochydore | North Lakes | Tamworth | Armidale www.covey.com.au

DOCUMENT ISSUE APPROVAL

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APPENDIX A	Ground Floor Structure - Removed Perpendicular walls

1.0 Introduction

1.1 Site Location

The wall "rear retaining wall" is located on the western elevation of the Suttons Beach pavilion building, ie;

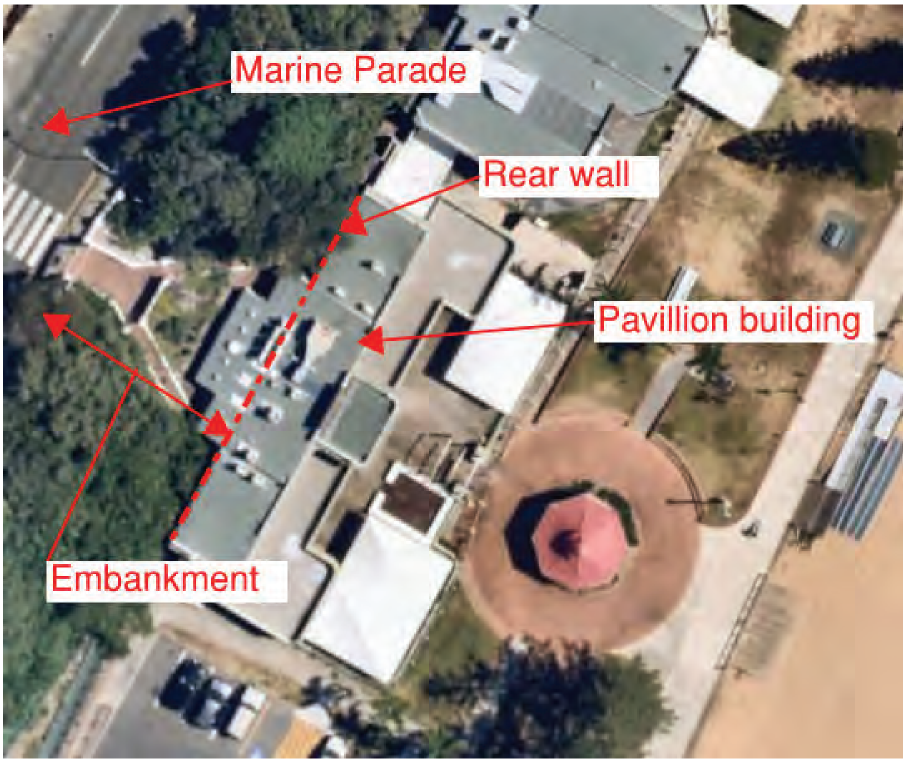


Figure 1 – location of rear retaining wall. The wall is (approximately) 37 metres long x 3.0 metres high.

1.2 Component description

Original concrete cast-in-situ retaining wall.

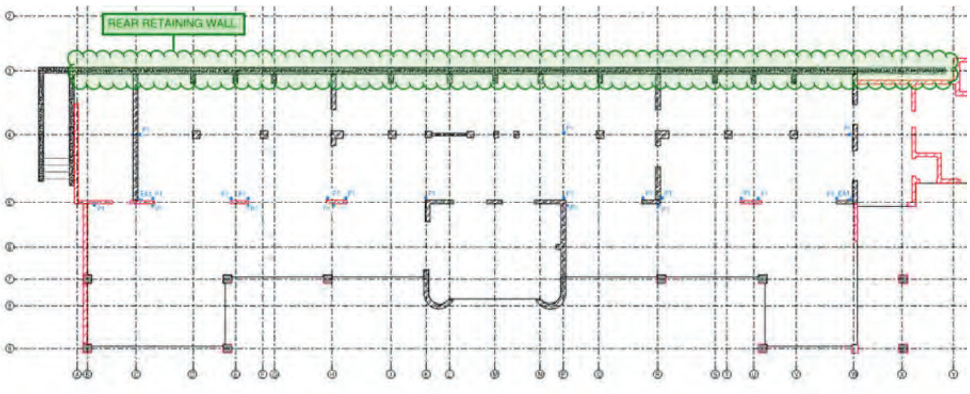


Figure 1: Plan showing extent of rear retaining wall.

1.3 Component reference

3-A/X

Concrete Retaining Wall running along Grid 3 between Grids A and X. (individual segments of wall between buttress columns and brick walls are labelled between gridlines e.g. 3-A/C denotes rear retaining wall segment along grid 3 between grids A and C).

1.4 Construction

- Wall assumed to be cast in situ with footing.
- Concrete buttress columns or nibs appear to be poured integral with the wall.
- There are also brick walls built perpendicular to the retaining wall. Some of these walls have been removed. See original versus current plans.
- Original slab on ground poured separately.
- Another topping slab of was installed later (possibly post 1975 or post 2000). This topping slab was also had a protective coating/lino in most areas.
- The suspended slab sits on top of the rear retaining wall and overhangs in some places.
- Outside the retaining wall an original spoon drain was discovered, buried under a layer of soil and another concrete slab/path. See sketch detail in figure 2 below.
- There was also an older concrete path in the same plane as the original spoon drain but with a soil filled gap between them.
- The newer top path looked like it had been sealed to the wall just below the original window openings (which have since been enclosed with bricks and rendered, but there are gaps especially at the top).
- The concrete wall was rendered internally along with the internal buttress columns and some brick walls. The render ranged in thickness from approximately 3 to 10mm. The top of the wall above the conc path externally was rendered with a different style likely after the top concrete path was installed. The wall below this was not rendered externally.

1.5 Structural Components and details

- The concrete wall is approximately 300mm thick.
- Currently the height of the wall from top of topping ground slab to underside of suspended slab is approximately 2850mm.
- Original slab on ground is approximately 150 to 180mm thick
- Topping slab thickness varies in thickness approximately 70mm
- The spacing of the buttress columns and perpendicular brick walls/columns range between 2 and 3m approximately.
- Concrete buttress wall dimensions are approximately 225mm by 450mm approx.
- Some original brick walls and concrete lintels were removed, leaving a brick buttress of similar size to above buttress. See Figure 5 below.
- The suspended slab design thickness varies between 150 and 200 originally but now is covered with a topping slab that varies from 15mm thick internally to approx. 75mm thick at the step line along grid 4 to even thicker (over 105mm) towards grids 5 to 8.
- Originally the suspended slab overhung the retaining wall – it extended past approximately 215 to 250mm.
- Sections of the overhang have been removed between grids G-H and R-W.
- The suspended slab appears to be continuous pour over grid 3 towards grid 2 between H and R.

1.6 Reinforcement

- All reinforcement scanning was carried out by others. See Appendix H in main report for results.
- Scans by others revealed no reinforcement in the rear face of the wall against the earth.
- Scans found reinforcement in the front face of the wall being ½ inch round vertical bar at 300 to 400mm spacing and ½ inch round horizontal bar at 300mm spacing.
- Scans found the cover to vertical bars was 60 to 85mm from the inside face. Scanning from the outside showed the cover to bars to be approximately 230-240mm from the outside face.
- Break out revealed vertical bars first at 50, 60 and 80mm cover then horizontal bars behind.
- Scans found 1 vertical ½ inch bar in each face of the concrete buttress columns and 3 horizontal bars (possibly ligatures) one central and then the others approximately 400mm above and below.
- Breakout was carried out to search for steel reinforcement tying the suspended concrete slab to the rear retaining wall. – None was found. Vertical bars exposed during breakouts had hooks finishing within the wall. See Photos in observation section below.

1.7 Geotechnical parameters

- A small section of the footing was exposed internally at 3-R/S and no toe was measured.
- Externally a section of the retaining wall was exposed at approximately 3-C/E. A resulted in Benching was required and a number constraints (physical, electrical, geotechnical and safety) meant only the top of the rear external footing could be exposed. See sketch summarizing findings in Figure 2 below.
- A geotechnical engineer carried out tests and was present for footing excavations including the rear retaining wall at 3-D/E. See results in Appendix G of Main report.
- The Geotech engineer stipulated active pressure conditions for the wall with
- Ka = 0.3 for Foundation/Footing material and
- Ka = 0.4 for Backfill behind the wall.
- The geotechnical engineer also recommended to consider hydrostatic pressure as a short-term load case behind the wall due to the possibility of water building up in a significant weather event.

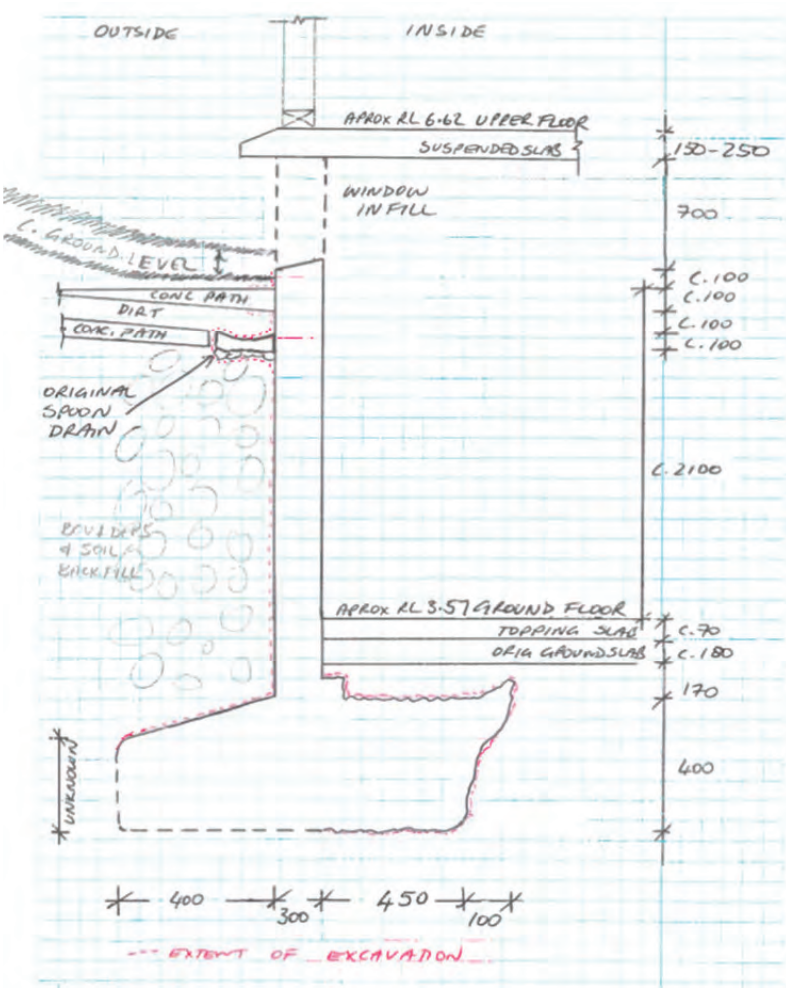


Figure 2: Sketch of retaining wall section

1.8 History

The history in terms of exposure and demolition is important when assessing the integrity of the structure from strength, serviceability and durability perspectives.

The Southern section of the rear retaining wall was originally exposed beneath the stairs. See figures 3 and 4 Below. Before being enclosed and used as a storeroom circa 2000 and then later refurbished as internal amenities.

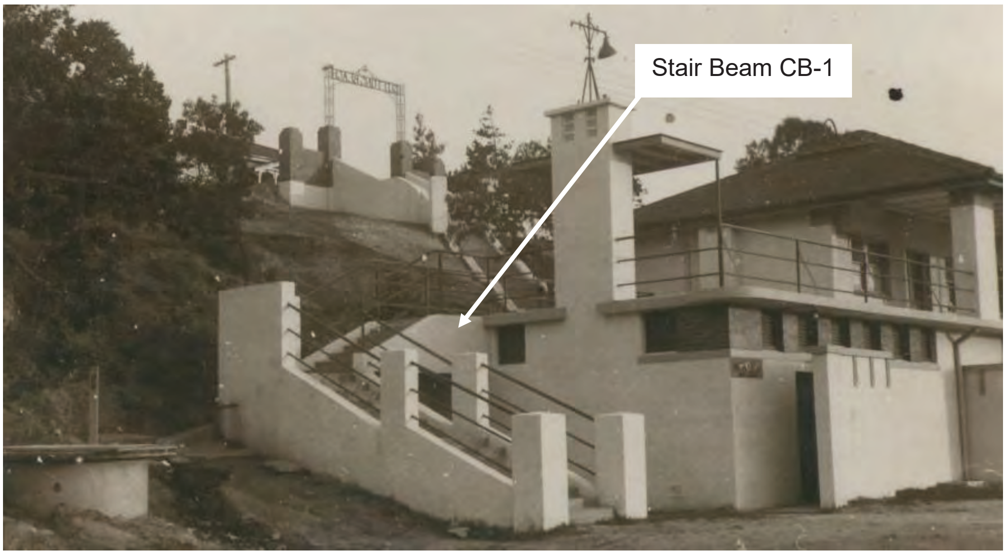


Photo 1: Southern side of original building circa 1938.

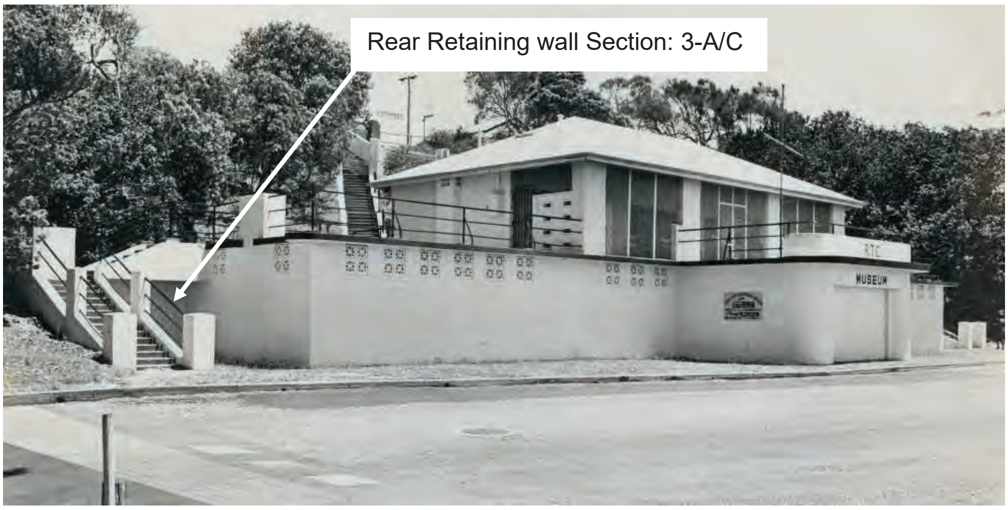


Photo 2: Southern side of Original building circa 1990.

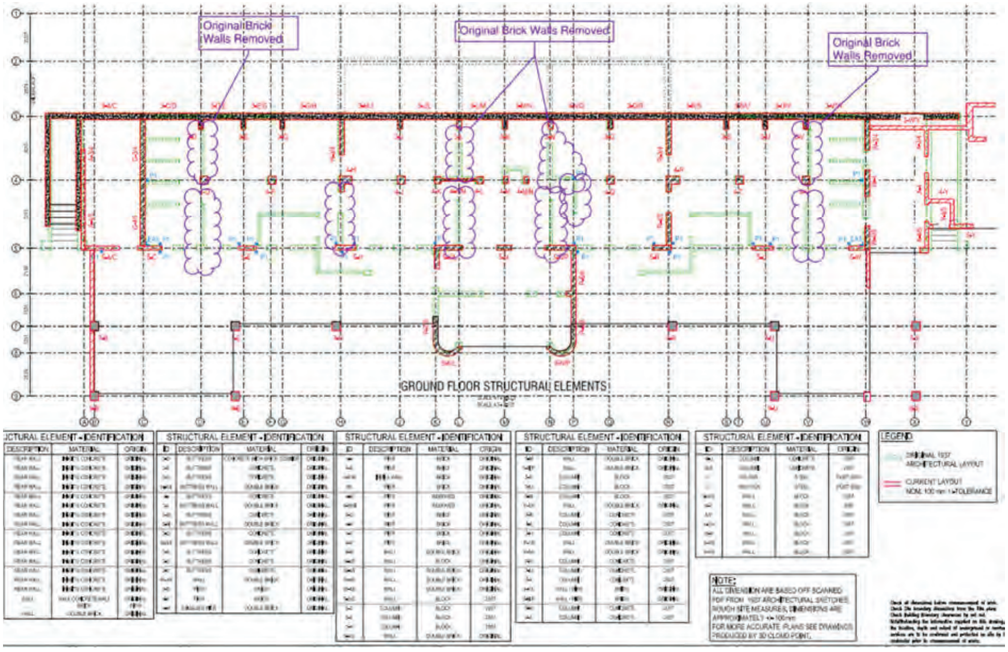


Figure 5: Removed perpendicular original walls clouded in purple. (see appendix for full size drawing)

2.0 OBSERVATIONS

Detailed observations, comments with supporting photos taken internally and externally section by section of wall.

When internal linings and cladding were removed – water and moisture ingress was evident as some of these linings were warped, damaged and mouldy, same with the non-loadbearing wall framing.

Photo 3: Drip lines and mould at crack in rear retaining wall.

See other pictures below.

2.1 Moisture ingress

The rear retaining wall is not lined/coated/waterproofed.

Moisture content (MC) readings were taken at random locations along the inside face of the rear retaining wall on July 28, 5 sunny days after a rain event.

Measured readings ranging between 16 and 33%MC were taken at the base of the wall. At mid height (approx. 1200-1500mm) the readings reduced in most areas to between 10 and 24 % MC. Readings nearer the top of the wall (2000-2500mm) had an even lower range of 0 to 10%.

There are a number of factors must can affect readings including but not limited to the relative humidity, variability in concrete mixes, the meter itself etc. So what is important to note is the MC decreasing from nothing/very low at the top of the wall (where there is no retaining and better ventilation/drying) to the much higher MC at the base of the wall (where retained material may not be draining sufficiently – leaving water sit behind the wall).

2.2 Section 3-A/C

This section of wall was originally exposed beneath the stairs for many years before the area was enclosed in for a store room and later for amenities. There is a lot of water ingress in this area, especially at the stair to new suspended slab join and the top of the stair wall (see photos below). The suspended stair soffit is spalling and the beam has significant spalling, corrosion of reinforcement with section loss (*see report section on Original Concrete Beams*).

Photo 4: Water ingress at corner 3:A-B

Photo 5: Water ingress at top of wall 3:A/C

Photo 6: Break out to reinforcement in wall 3-A/C.

2.3 Section 3-C/D

Externally the wall retains around 2m. A timber deck/path runs adjacent the back of the building through this section. There was a lot of debris – soil/leaf matter/rubbish/build up under this deck and above the concrete path/apron. The top concrete path had obviously dropped as there was a gap in the previous waterproofing (*black jack painted sealant*) that showed. Furthermore, this waterproofing was no longer sealing the joint between the wall and the external path slab.

Photo 7: Rear retaining wall section 3-D/C.

Photo 8: Rear retaining wall section 3-D/C/B.

Photo 9: Rear retaining wall at approx. 3-D. Note gaps above window infills, spalling of concrete overhang, joint between suspended slab and wall.

Photo 10: Rear retaining wall section 3-E/D. Note the rough surface of the concrete wall externally, with areas of boney concrete, cracks and timber form imprints. Also, the original spoon drain must have been poured directly over the original free draining backfill boulders.

Photo 11: Shows breakout from internal face to steel reinforcement at 3:P.

Photo 12: Shows breakout from internal face to steel reinforcement at 3:C-D.


Photo 13: Photo at top of Butress column/wall at grids 3:D. Southern Face.

Photo 14: At top of Butress column/wall at grids 3:D. Northern Face. The column is mostly concrete with a brick corner that used to be a continuous brick wall between the rear retaining wall on grid 3 and the column on grid 4.

2.4 Section 3-G/H

Externally the wall retains around 2m. A timber deck/path runs along this area. There was a lot of debris – soil/leaf matter/rubbish/build up under this deck, to the extent that it was above the original window sills and concrete path/apron.

Photo 15: Rear retaining wall section 3-D to H.

Photo 16: Excavation behind retaining wall near grid 3:D/G.

2.5 Section 3-H/R

This section of wall supports the continuous slab above which is thought to be originally a slab on ground between grids 2 and 3 and suspended between grids 3 and 4. The slab on ground between grids 2 and 3 was dug out at some point and became a suspended slab. Thus retains approximately 1.5 to 2m. less retaining behind it as it looks like it was dug out to form an undercroft beneath the upper floor slab on ground. Externally the wall retains around 2m. A timber deck/path runs along this area.

Photo 17: Rear retaining wall between grids M and N. Note the hole in the block infill to the undercroft.

Photo 18: Taken inside undercroft area.

Photo 19: Top of wall 3:H-J.

Photo 20: Wall 3:H-J is gouged out, cracked across the top at the join to suspended slab.


Photo 21: Rear retaining wall. Note the cracks in the render were ground back and are reflected in the concrete substrate.

Photo 22: Cracks in render ground back show that crack is also in concrete wall and buttress substrate.

Photo 23: At 3-G/H Wall ground back shows cracks visible in render are also in wall below. Note both horizontal and vertical cracks on inside face – showing tensile capacity of concrete wall under load has been passed. Wall is operating as a cracked section.

Other vertical cracks approximately mid span between the buttress columns/gridlines were noticed in other bays. This could indicate tension crack on internal face.

2.6 Section 3-R/W

Originally a mirror image to the other end of the building (i.e. similar to 3-A/H). Note that it appears that 3 of the 4 steel beams supporting the slab are actually connected to the brick infills of the original windows. See more details below with the sections of wall between R and W as broken up with photos below.

2.7 Section 3-R/S

This section of wall originally had windows that have been filled in and grouted, but later the vents were installed and newer bricks grouted in around the penetration. However, this penetration was not sealed or water proofed and when the ground level outside beneath the timber decking was not maintained, it built up with leaf matter and debris over time so that the ventilation shafts were buried or half buried.

Photo 24: Vent penetration in rear retaining wall between grids R and S.

Photo

Photo 25: Rear retaining wall between grids R and S. Note the previously repaired vertical crack as well as horizontal cracks in the render.

2.8 Section 3-S/T

This section of wall has a similar history to that above. It should also be noted that the steel beam supporting the slab appears to be connected to the brick infill in the original window.

Photo 26: Rear retaining wall between grids S and T.

2.9 Section 3-U/V

This section of wall has a similar history to that above where brick infill was used to fill the original openings, the vent openings have gaps and are not sealed. It should also be noted that the steel beam supporting the slab appears to be connected to the brick infill in the original window. The vent openings have gaps and are not sealed. The wall is not connected to the suspended slab.

Photo 27: Rear retaining wall between grids U and V. Note the vertical bar on the left cogs over below the slab level. Also note the exposed slab reinforcement due to the slab soffit falling down when the layer of bricks directly beneath it was removed.

2.10 Section 3-V/W

Photo 28: Rear retaining wall between grids V and W. Note darker grey grout on the brick infills in the original windows.

Photo 29: Close up of the above vent penetration in rear retaining wall between grids V and W. Note the vertical bar at buttress column corner is hooked and does not extend into the slab.

Also note the steel PFC support the slab is bolted to the brick window infill immediately adjacent to the vent penetration. These bricks have also been cut to fit the vent.

Photo 30: Corner at 3:W Shows severe termite damage and wet rot to non-loadbearing wall framing timber. Also note the horizontal crack has water marks around it.

Photo 31: Close up of the above photo showing termite damage, wood rot and mould.

Wall to slab joint cracked in many places:

Photo 32: Cracks in top of retaining wall at join to suspended slab and brick wall at 3:R Southern side.

Photo 33: Butress at 3:Q Northern side.

Photo 34: Butress at 3:Q Southern side.

Photo 35: Removed brick wall at N-3/4

Photo 36: Removed brick wall at L-3/4. Note crack along the rear retaining wall to slab join as well as crack in slab and core hole through slab.

Photo 37: At 3-H/J Note cracking along top of what's left of the rear concrete retaining wall.

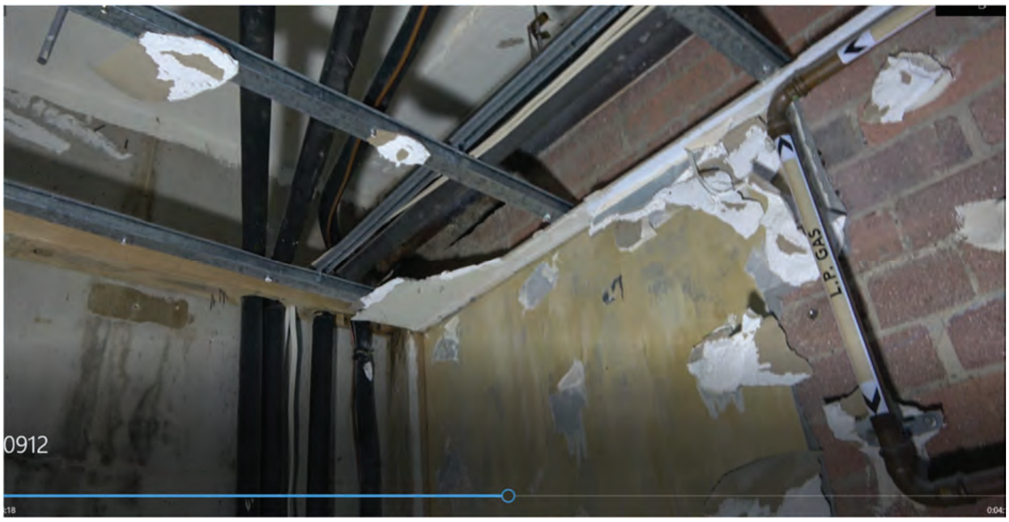


Photo 38: Wall at 3-G to H. Note horizontal cracks in rear retaining wall and mould. Also, perpendicular brick wall along grid H.

Photo 39: Cracking in rear wall (close up from above photo) at grid 3:H.

Photo 40: Cracking at 3:G in slab at buttress/wall.

Photo 41: Crack/gap between slab and retaining wall all the way between 3:D-C.

Photo 42: At 3:C-D.

Section 3-W/Y:

This section of the rear retaining wall was originally part of the suspended stairs but they were cut off leaving reinforcement exposed (possibly in 2001 renovations). This wall is in the "*Link Area*". Part of it was exposed beneath the stairs for many years and was originally supported by the suspended stair, landing and the perpendicular section of the stairs. This wall is exposed to the weather.

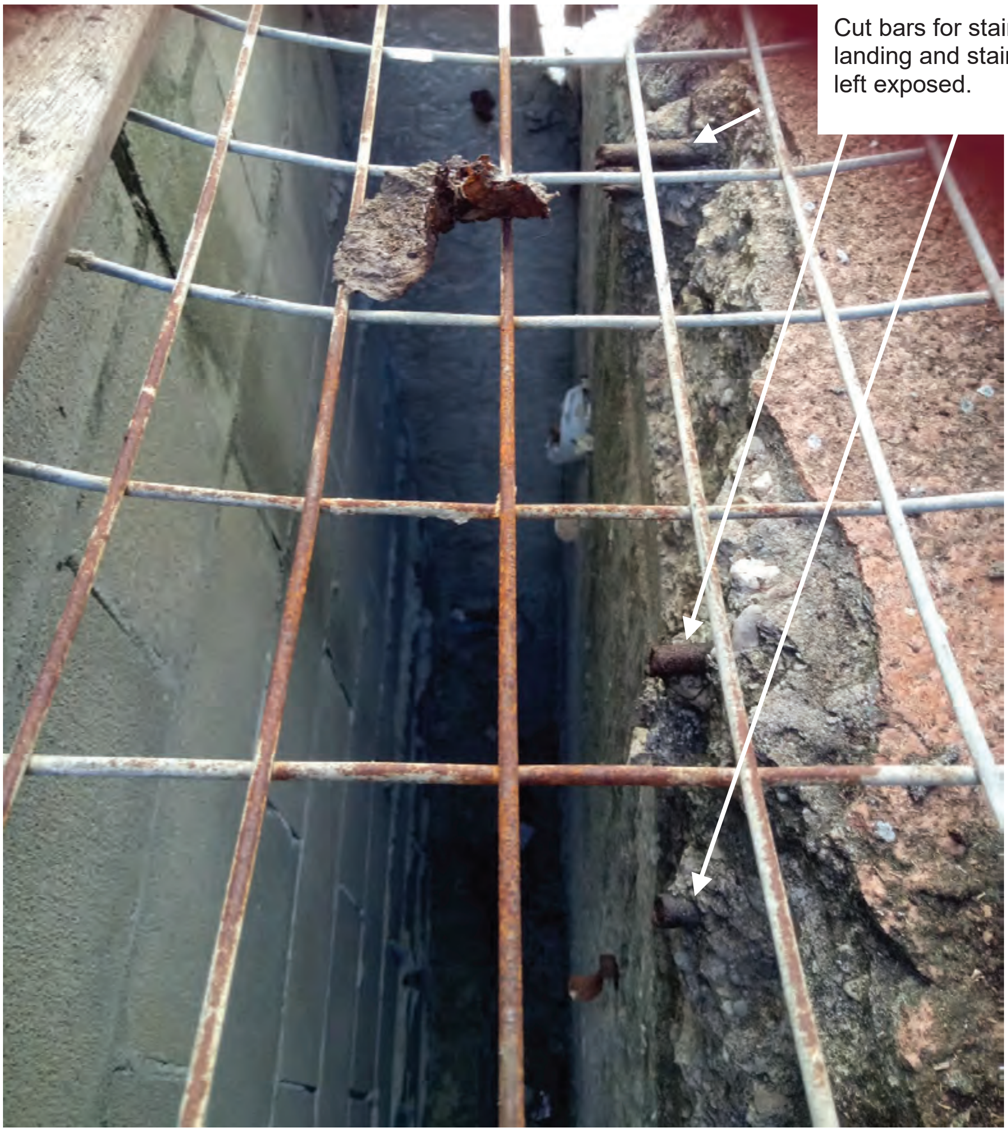


Photo 43: Top view looking down at original rear retaining/stair wall on the right. Note cut exposed reo.

3.0 Summary of issues

1. **Wall cracks:**
Horizontal cracks approximately 1/3 and 2/3 wall height run mostly the full length of the retaining wall. Other horizontal cracks and vertical cracks are present as well.

These cracks were showing in the render. At a number of locations the render was ground back to confirm that these cracks were reflected in the substrate (concrete wall or buttress). It is not known if these cracks continue through the entire thickness of the wall as the external side was not able to be checked. Although at the vac excavation the external surface appeared quite rough with many defects and timber board form markings see photo 10.

2. **Perpendicular support walls removed:**
All or part of the perpendicular along grids D, L, N, V and W were removed. See plan in appendix A for clarification.
3. **Rear wall drainage unknown:**
There is currently no drainage behind the retaining wall. Large boulders in dirt may have originally been free draining but now are filled with fines. No agg pipe, geofabric or outlets were found during the vac excavation behind the retaining wall.
4. **Insufficient surface water drainage grids A-G:**
Stormwater drainage – cracked and leaking pipe.
Multiple concrete layers with soil between them do not divert water away from the building.
There is also an unsealed gap at the join between the top conc. This would allow surface water ingress directly behind the wall as well.
There is limited space beneath the timber deck walkways making it nearly impossible to maintain the surface and keep it free of leaf matter/rubbish/etc.
5. **Insufficient surface water drainage grids R-X:**
Ground level above vent openings.
This allows surface water to pour into the building at the vent penetrations.
Again, there is limited space beneath the timber deck walkways making it nearly impossible to maintain the surface and keep it free of leaf matter/rubbish/etc.
6. **Lining behind wall:**
There is no waterproofing or lining behind the retaining wall. It is bare concrete with no render and hence any water sitting against this wall could make it's way through any cracks or boney sections of concrete especially if under pressure. During site visits no water was seen coming through the walls, but moisture measurements were taken and were much higher at the base of the rear retaining wall.
7. **Moisture ingress through wall:**
Internal moisture readings were still high in bottom third of wall even after 5 sunny days post rain event. Indicating water is not draining away quickly. This has affected and will affect any future wall framing/linings/cladding.
8. **No connection of wall to suspended slab:**
There appears to be no connection between the top of the retaining wall and the suspended slab. Vertical bars were exposed at a few locations and they stopped and hooked over within the wall and did not extend across the joint into the suspended slab.
9. **Rear wall inaccessible at undercroft:**
Enclosed space. Cannot excavate behind wall or could undermine the higher level footings along grid 2.
10. **Infill sections unknown:**
Old windows were infilled with bricks – it appears that some of the steel beams supporting the slab are connected directly to these brick infills.
Also, sections in the retaining part of the wall were removed and filled in with blockwork - the reinforcement and dowels to existing are unknown.
11. **Other issues are mentioned in the BG & E testing report.**
12. **Wall section 3:W-Y**
Used to be stair wall, but was cut exposing reinforcement and is still retaining but has no more support.

4.0 Structural assessment

4.1 General

ACOR Review – From site inspections we note that depth of retention is approximately 2.1 metres.
To the west, Suttons Beach Pavilion is bordered by an embankment, immediately adjacent to Marine Parade in Redcliffe. The building is significantly lower than the adjacent road level.
To the east, the Pavilion adjoins the Moreton Bay foreshore.

Given the site conditions, there is currently little (or no) opportunity for relief of hydrostatic build up in times of high intensity (or any) rainfall. As a consequence, when stormwater accumulates behind the wall, the resultant hydrostatic pressures act, in addition, to the normally retained earth pressures.

Photo 1 – level of retention at rear wall.

Note, the concrete element visible against the wall in photo 1 are actually the remainder of the cut path and spoon drain.

Generally, the level of the retained ground was at, or about, the base of the window level. The dirt marks (against the yellow wall) can be seen in the photo (1).

4.2 Design Parameters & Assumptions

We acknowledge a substantial level of site investigation, testing and reporting that, in addition to our own attendance, has been done by other consultants.

Notwithstanding, our collective site investigations suggest the following:

1. That there is no connection, that could be located by extensive investigation, between the top of the retaining wall and the level 1 slab.
2. That we are unable to confirm any connection between the footing and the base of the retaining wall.
3. Generally, that the height of the existing ground level, being retained by the wall is 2.1 metres, with a very steep slope behind (see further comment).

<u>Comment about retained height</u> - Referring to figure 9, it can be seen that there is an amenities area located to the west of the retaining wall between grids H & R.

The under croft (ie; the area under the amenities slab, see photo 4) is largely void.

Being a confined space, access to this area was extremely limited. However, the photos that we were able to take suggest that there is little retained earth (against the wall) in this area.

With reference to the BG&E assessment report (Aug 2022), section 7, we accept their advice and adopt the following design parameters:

- Wall thickness ~ 300mm
- Concrete strength ~ N25
- Vertical reinforcement ~ R12 @ 300-400mm crs
- Horizonal reinforcement ~ R12 @ 300mm crs
- Reinforcement all oriented towards front (internal) face of wall
- Cover from front face of wall ~ 60-85mm
  <u>Commentary</u> - for the purposes of structural analysis, we have defaulted to the more conservative value, ie; 60mm. We do note that at location grid P, the actual scanning returned a measurement 50mm cover.
- Cover from rear (retaining) face of wall ~ 230mm
- Reinforcement only in front face of wall (ie, nothing towards retained face)

In the absence of specialist advice to the contrary we have assumed, for reinforcement, a yield stress (Fsy), of 250 mPa.

Regarding reinforcement, from site investigations, we note that both vertical and horizontal bars in break out at 3 (between grids A & C) were ½ inch bars.
The bars at breakout 3 (between grids C & D) were also exposed. We note that front vertical bar was approximately /8 inch bar (see photo 2) This doesn't affect the strength calculations, as the observations suggest that the wall is spanning horizontally. The horizontal bars are definitely ½ inch (ie; ~12mm).

Photo 2 – reinforcement at breakout 3 between grids C & D.

4.3 Geotechnical Parameters

Core Consultants geotechnical report (15 Oct 2022), page 5, paragraph 6, includes:

The rear retaining wall is retaining the stiff to very stiff residual soils with a significant backfill zone, the earth pressure coefficient for the residual soils would be 0.3, but 0.4 for the backfill (given the size of backfill zone this value is suggested for the wall about 0.9m (above footing) for design checks. As mentioned, there were coarse cobbles behind the wall but not evidence of a pipe, so the outlet for the cobble backfill is unclear and this should be clarified and rectified, or provision for water pressure is recommended.

There are two (2) earth pressure co-efficient mentioned, 0.3 & 0.4.

The geotechnical report does not make the distinction between 'active' and 'at rest' pressures.

Given that the wall is part of the greater structure, we suggest that the assessment of its ability to retain should be based on 'at rest' forces. Additionally, in a practical sense, it's been there for years.

In other circumstances, we would expect an earth pressure co-efficient for ‘at rest’ forces to be somewhat greater than those suggested above. However, we would default to the geotechnical engineer’s advice in this matter.

Further, there is a steep slope immediately to the west of the retained earth. The geotechnical report does not specifically mention the slope or its effect on the retained materials. We are not aware whether the slope consists of the natural hard clay or rock. By observation, the slope seems rather stable.

We also note that this geotechnical report is a “draft” copy.

Given the above, for the purposes of this report, we have defaulted to the more conservative 0.4 value for our structural revue.

However, that may be revised on receipt of further geotechnical advice.
Our independent assumptions include:

- In-situ soils bulk density ~ 18 kN/m^3
- Water bulk density ~ 10 kN/m^3

Again, these assumptions need to be confirmed by the geotechnical engineers, as they are the experts for this site.

4.4 Further Observation

As a further observation, vertical cracking was identified approximately mid-way between the rear wall nibs/buttresses, in many bays.

Further to our earlier assumption that the wall is spanning between nib walls, this manner of cracking likely indicates tension cracks, for the horizontal spanning wall.

Although not quite obvious, the attached photo (3), taken at grid 3:S/U, is somewhat representative.

Photo 3 – vertical cracking at grid location 3:S/U

There were more of these types of cracks between grids A-H as well.

Additionally, there are horizontal cracks running the length of the building at approx. 1/3, half way and 2/3 of the way up the wall.

All of the cracking (described here) is on the inside wall rendered face.

4.5 Comment on crack control

Given the quantity of reinforcement identified in “Design Parameters & Assumptions”, and with reference to AS3600:2018 Concrete Structures, clause 11.7, the Code requires the minimum area of reinforcement for shrinkage (crack) control to be 0.006.

We know that the area of steel is generally (the equivalent of) 12mm bars @ 300 crs, and the general wall thickness is 300mm.

The actual reinforcement ratio achieved is given by the following equation:
367mm^2/m (reinforcement) / 300 x 1000 = 300,000mm^2/m (concrete) = 0.0012

Therefore, there is 0.0012/0.006 ~ one fifth (1/5) the quantity of reinforcement within the wall that the Code would normally require for this location (exposure classification).

In summary, the wall is (approximately) 80 years of age. All of the cracking due shrinkage occurred long ago.

The cracks described above may have resulted from initial shrinkage, or they could be the result of non-compliant wall strength (discussed in section “Analysis results”).
In any case they are small and don’t affect the intent of this report.

4.6 Geometry

For design purposes, we have assumed a conventional prismatic distribution of active pressures/forces acting upon the wall.

The following figure (2) indicates the forces adopted for design purposes, and the heights at which they act against the wall.

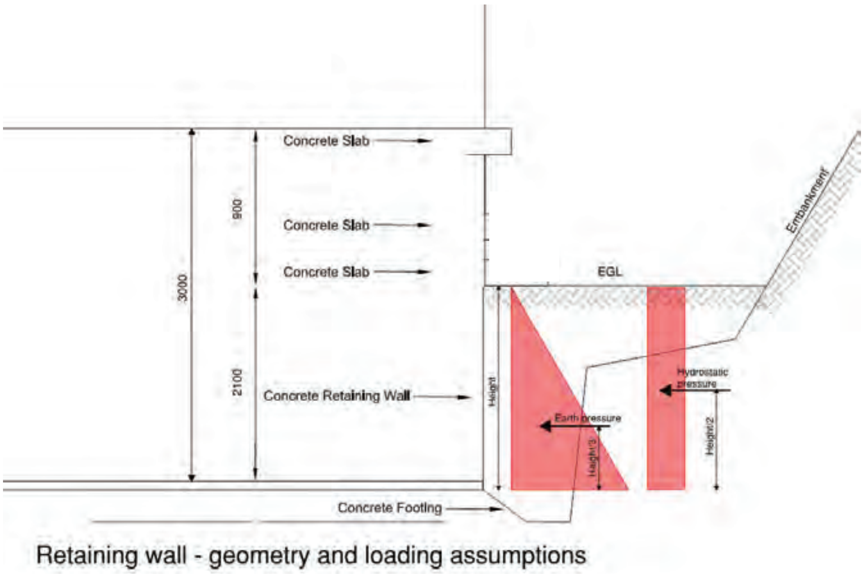


Figure 2 – forces acting on the wall.
Noted – a triangular distribution for earth loads is assumed, for the purpose of reporting. This needs to be confirmed by the geotechnical engineer.

4.7 Distribution

To rationalise the distribution of forces on the wall, the following figure (3) can be used to articulate the comparative magnitude of forces at the varying heights.

As an explanation:

- Within the lower third of the wall (Zone A) the forces are greatest,
- In the middle portion of the wall (Zone B) the forces are somewhat less, and
- At the top part of the wall (Zone C) there are no earth of hydrostatic pressures acting against the wall.

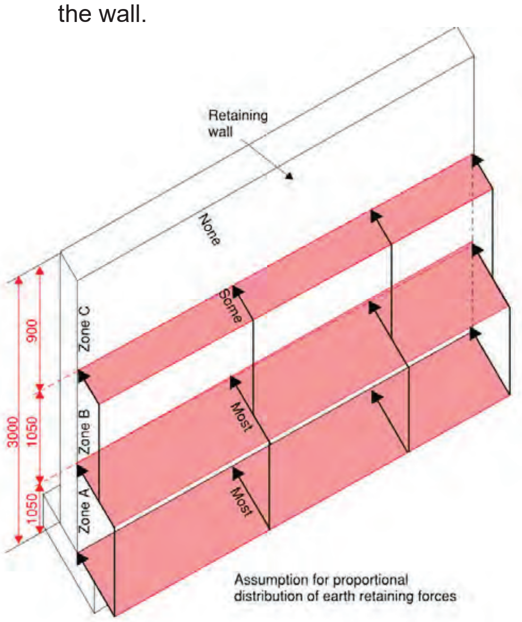


Figure 3 – assumed proportional distribution of earth retaining forces

4.8 Wall restraint conditions

1. Support by vertical (abutting nib wall) restraints only.
   As discussed above, (extensive) site investigations uncovered no evidence of reinforcement connection between the retaining wall and either the strip footing/ground slab below or the level 1 slab. Therefore, for the purposes of analysis, in its simplest terms, we would assume that the wall is spanning horizontally and is restrained by the exiting abutting nib walls, ie; a 'one way' span. This is the most conservative approach.

2. Restrained on four (4) edges.
   If the wall could be effectively restrained at its base and top (as well as the existing vertical restraint described above). This is a less conservative approach.

   The two (2) conditions are illustrated in figure 4, below.

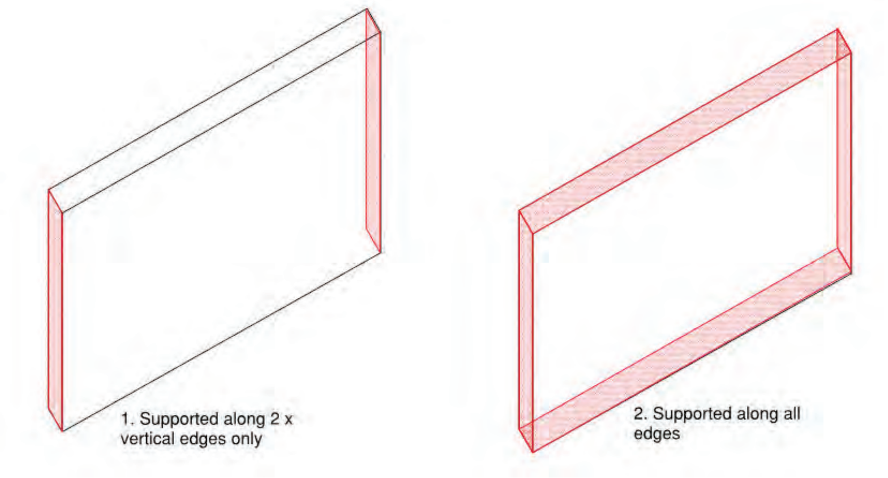


Figure 4 – wall restraint conditions.

4.9 Analysis results

We have analysed the wall for structure, for both restraint conditions, against the requirements of AS 1170.0:2002 “Structural design actions, Part 0: General principles” and AS 3600:2018 “Concrete Structures”.

For condition 1, our results are presented in the following figure (5).

Span (mm)	At rest forces	Zone	Retained depths (m)	Result- wall capacity at supports	Result - wall capacity at mid span
2780	Earth + Hydrostatic	A	2.1m - 1.05m	Fail	Fail
2780	"	B	1.05m - 0	Fail	Pass
2000	Earth + Hydrostatic	A	2.1m - 1.05m	Fail	Pass
2000	"	B	1.05m - 0	Fail	Pass
2780	Earth only	A	2.1m - 1.05m	Fail	Pass
2780	"	B	1.05m - 0	Pass	Pass
2000	Earth only	A	2.1m - 1.05m	Fail	Pass
2000	"	B	1.05m - 0	Pass	Pass

Figure 5 – results of capacity of retaining wall to accommodate induced bending moments.

4.10 Combined earth and hydrostatic pressure

Generally, when both earth and hydrostatic pressures are acting in combination, the wall fails. The predominant failure mode is at areas of negative bending over support locations.

Specifically, the main failure locations are:

1. Within the lower and mid portions of the wall, Zones A & B (ref Fig. 3), the wall failed in negative bending at the supports (at all locations). That is, the tension developed towards the rear face of the wall is beyond the ability of the un-reinforced concrete to accommodate.
2. Within the lower portion of the wall, Zone A (ref Fig. 3), the wall failed in positive bending within the larger (2780mm) span. That is, the quantity of reinforcement within this mid-span area of the wall does not afford the wall the ability to accommodate the resultant stresses.

However, when we assumed that the wall was restrained on four sides (restraint condition 2), in all zones, the results of our structural analysis suggested that the cracked bending moment (Mcr) was either closer to or exceeded (depending on location) the design bending moment (M*).

Therefore, the 300mm thick wall will have considerable moment capacity, ***without*** reinforcement, provided it stays below Code limits, and it does not crack.

In summary,

- If the wall is restrained along two (2) edges only, the analysis suggests that it does not conform to Code limits and will likely require substantial upgrade to achieve an acceptable solution.

- If the wall is restrained along four (4) edges, the analysis results more closely mimic the required Code limits, and the wall will likely require substantially less upgrade to achieve an acceptable solution.

4.11 For the case of earth pressure only

When assessing the wall for earth pressures only, with the exception of the negative bending area at the Zone A longer span supports, the wall proved compliant.

More specifically, our analysis results suggest that:

1. Within the lower portion of the wall, Zones A (ref Fig. 3), the wall failed in negative bending at the supports at all support locations. That is, the tension developed towards the rear face of the wall (over the supports) is beyond the ability of the un-reinforced concrete to accommodate.

2. At all other locations, for this load case, the wall proved compliant.

In Summary:

- Under the combined cases of earth and hydrostatic pressures, the wall is largely non-Code compliant.

- If the wall was relieved of the hydrostatic pressure, apart from the areas identified above, it could be considered largely Code compliant.

Therefore, in terms of structure, for the wall to be considered serviceable, it needs to either:

- Be relieved of the hydrostatic pressures and be remediated to address those areas identified above, or

- Be remediated to accommodate the combined pressures and address the structural deficiencies.

In terms of amenity, as identified by others, the wall has to be waterproofed, sufficient preclude the ingress of either ground or storm water in order for the building to be fully utilised.

4.12 Structural analysis of end stair walls

By Covey - Wall Section 3:W/Y (Old Northern stair rear wall or *Link area*)

After the stairs were cut off there was no more support for this wall between grids W and Y. Following on from the analysis from the rest of the wall it does not have adequate strength to meet the current design standards with required safety factors. Furthermore, the cut reinforcement was left exposed and is corroding.

Structural analysis of the wall as a cantilever shows that it fails in bending for cases 1 and 2.

Wall strengthening is required as part of remedial works.

Wall section 3:A/C has perpendicular support walls and is to be treated like the main rear retaining wall 3:C-W for structural strengthening.

4.13 Serviceability and Durability Comments

The intrusions of moisture through the concrete rear retaining wall exposes any internal framing/cladding/coatings in direct contact with the wall. In addition, lack of ventilation promotes conditions for the growth of mould, wet rot, peeling coatings etc. Hence any new non-structural framing/cladding to internal rear retaining wall will be at risk if the wall is not waterproofed.

Although wall section 3:W/Y was not accessible it is expected that this wall is at the end of it’s service life as reinforcement has been exposed for many years and with the removal of the stairs and reo left exposed it’s condition would be worse than 3:A/C. Refer BG&E Report for details on 3. It requires remediation.

5.0 Remedial discussion

By ACOR - To render the wall fit for purpose, our investigations suggest that there are a number of areas that need to be addressed, ie;

1. Relieve the pressure on the wall,
2. Conduct remedial works sufficient to render the wall compliant for structural effects,
3. Tie the abutting brick nibs to the wall,
4. Tie the level 1 slab to the wall, and
5. Render the area waterproof.

If no discernible remediation works are undertaken, and with reference to the B G & E report (August 2022), clause 7.5, “the wall is considered at the end of its effective serviceable life”, the area will continue to deteriorate, permit water ingress and generally contribute to the genera deterioration of the structure.

5.1 Pressure relief

To minimise the commitment of resources to the remedial process, the most immediate course of action is to relieve the pressure on the wall.

This is most efficiently done by removal of the hydrostatic accumulation, from whatever source (ground water, storm water or leaking services).

We note that with regard to possible remedial works a number of past reports have been prepared, including:

- Pumped de-watering system, Vosloo Consulting Engineers, Jan 2017
- Sub surface drainage and additional wet wall, FA Consulting Engineers, March 2017
- Sub surface drainage and additional wet wall, FA Consulting Engineers, June 2017
- Dewatering, FA Consulting Engineers, June 2019

Also, we note that the neighbouring property has constructed a substantial concrete crib wall at the western embankment (of their property).

Any solution will likely require substantial excavation from behind the wall. Construction considerations will include (but not be limited to) safe access, retention of the unexcavated area, general slope stability and drainage. These (and other) issues have been highlighted within the referenced reports.

Further, for the area between grids H to R, the existing amenities rooms are built over. We were able to look within the crawl space (ie; under the amenities slab and beyond the rear retaining wall during our inspection. The following photo (4) is taken within this space and shows the footings supporting the upper-level amenities area slab, ie;

Photo 4 – Under croft of amenities room over between grids H & R

The significance of this is that any proximate excavation (to the retaining wall) would need to be done in such a manner that didn’t undermine this part of the structure. Again, this would require specialists’ advice.

5.2 Strengthening

Strengthening may involve either:

- Remediation of the existing wall, or
- Creating a new wall, that is separate to the existing.

Our review suggests that the areas deficient in strength, are those areas at supports. As advised within the BG&E report, there is no reinforcement near the tension face at these locations.

While our analysis results suggests that the cracked bending moment (Mcr) capacity (of the wall) is within Code limits, all of the assumption used in performing that analysis need to be confirmed (or otherwise) by the relevant project experts.

If the intention is to remediate the wall in-situ, and if additional reinforcement is required, consideration could be given to the use of carbon fibre strip. Such techniques are commonly used in remedial situations. The carbon fibre is glued to the face of the concrete wall and mimics the tension restraint that would otherwise be afforded by the reinforcement.

The carbon fibre strips would likely be applied as indicated in the following figure (6), ie;

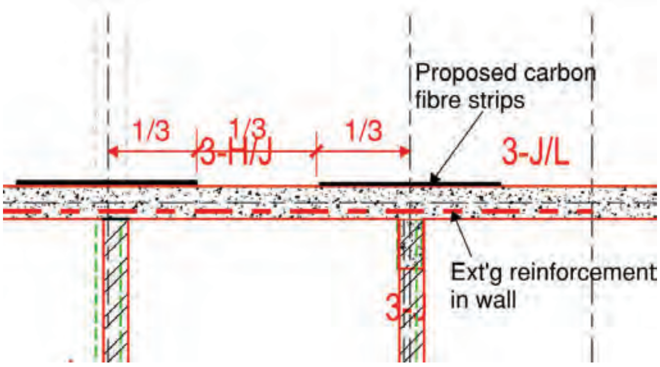


Figure 6 – Plan - likely location of carbon fibre strip

Being located on the back (retained) side of the wall, these strips would need be protected (from soil, water, etc).

If the intention was to augment the existing retaining wall with an additional compliant wall, we suggest that either reinforced concrete masonry units, or a reinforced concrete structure would be appropriate. Such a wall could be constructed inside or outside of the existing wall.

If it was constructed inside the wall, it would not be necessary to excavate behind the wall, for structural purposes.

However, individual panel lengths would be dictated by the distance between the abutting masonry nib walls. In this case the new wall would need to be attached to the existing masonry nibs by concrete anchors, dowels or some other form of restraint, all of which would serve to intrude on the existing masonry element.

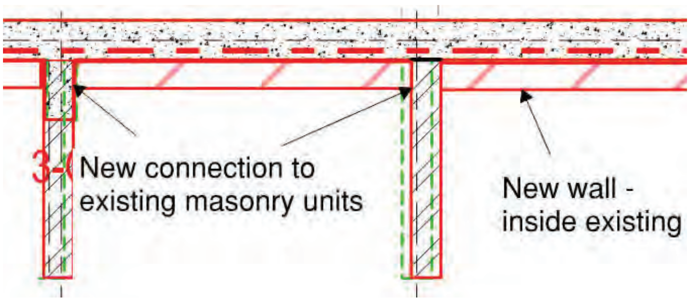


Figure 7 – Plan - indicative “inside” new wall

If the new wall was constructed on the retention (out) side of the existing wall, the process would require significant excavation, as alluded to in the previous section (of this report) and the referenced past reports.

Conversely, location of the new wall on the retention side would have significantly less impact on the heritage structure, and would also allow for waterproofing works (ie; to the back of the wall) to be done simultaneously.

Our site investigation suggests that the existing footing is wide enough to accommodate a wall of 200 – 3300mm thickness. However, the design would be contingent on confirmation of the capacity of the footing by a geotechnical engineer.

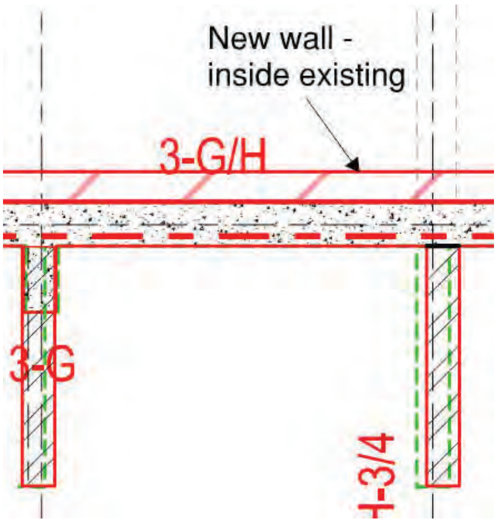


Figure 8 – Plan - indicative "outside" new wall

5.3 Waterproofing

Effective and efficient waterproofing is a specialised discipline within itself. As structural engineers, we would suggest that specialist's advice be sought for this application.

Notwithstanding, to apply a waterproofing product to the retained (out) side face of the wall, as alluded to earlier, significant excavation, that extends from the ground level to the footing, would need to be done to allow access to the area. Further, the excavation would need to be wide enough to permit the required materials, machinery and labour.

6.0 LOWER LEVEL BRACING

ACOR Review – Suttons Beach Pavilion – ***Lower-level bracing.***

6.1 Current situation

Review of the Covey drawing (figure 9) reveals there are a number of nib walls abutting the rear retaining wall, ie;

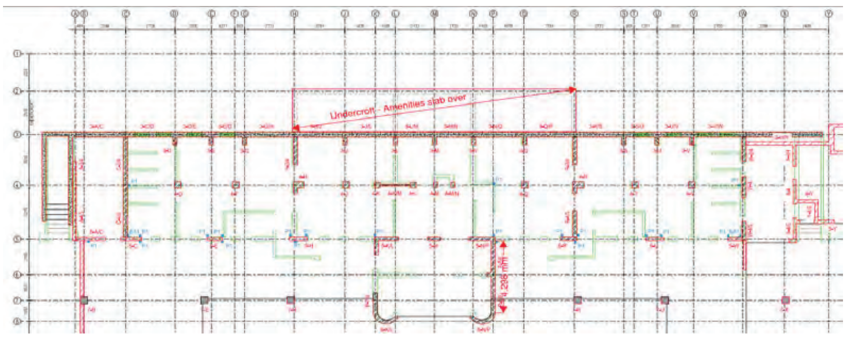


Figure 9 – Covery drawing "Ground floor structural elements".

Some of these nibs are masonry, some are partially reinforced concrete and other are a combination of both.

By inspection we note that, without exception, these nibs were likely part of a longer wall. The extent (length) of these walls has been amended to the current (very small) length, as part of successive renovations of the structure.

Our review suggests that these nib walls, as they currently exist (see Figs. 4 & 5) ***are not*** adequate to accommodate the lateral forces generated by the earth retaining pressures.

Photo 5 – existing nib walls

As previously noted, we suspect that successive renovations of the premises have severely curtailed what was likely, the original length of the walls (see figure 9 for current wall layout).

6.2 Design

Regarding resistance of lateral forces, the following figure (10) indicates our loading and design assumptions. It also indicates the (approximate) length, to which a bracing element will likely need to extend, to provide adequate resistance.

This is based on simple static analysis.

Figure 10 (below) indicates the likely extent of the remediated bracing elements.

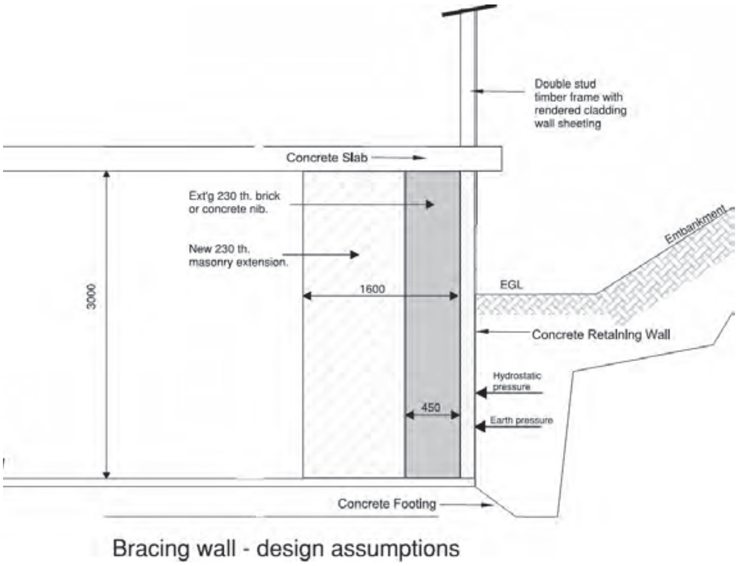


Figure 10 – bracing elements

Such bracing elements may be double skin brickwork, reinforced concrete, structural steel frames or a combination of any or all, as best suits the proposed building amenity at the time of design.

Notwithstanding, the 1600mm dimension represents the effective depth of the element.

For example:

- At brickwork nibs, the new brick panel will need to be "toothed" to the existing, and
- At concrete nibs, the new bracing element will need to be attached with appropriate masonry anchors

6.3 Locations and extent

Figure 11 (below) shows the locations and extent to which the walls are required, to provide bracing.

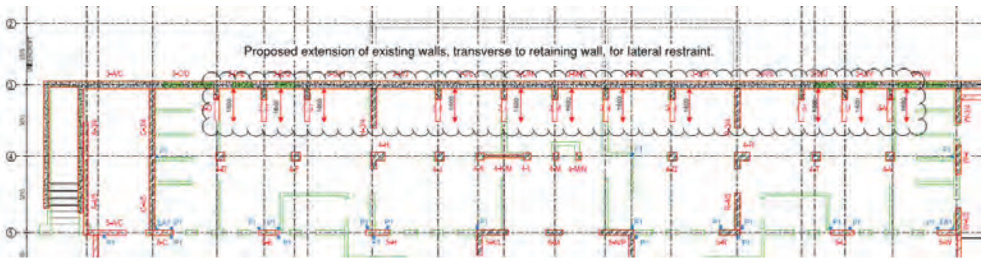


Figure 11 – indicative bracing required to resist earth generated lateral forces.

Further to those considerations discussed above, the bracing elements need to:

1. Be anchored to the ground floor slab. This would likely be achieved by masonry anchors. The choice of bracing type (masonry, concrete od structural steel) will need to consider the manner in which this connection is effected.

2. Be restrained at the soffit of the level 1 slab for lateral and shear effects. This could be achieved with the use f built in masonry anchors (within), or steel angles and masonry anchors attached to the side of the bracing wall.

These restraint conditions are indicated in the following figure (12), ie;

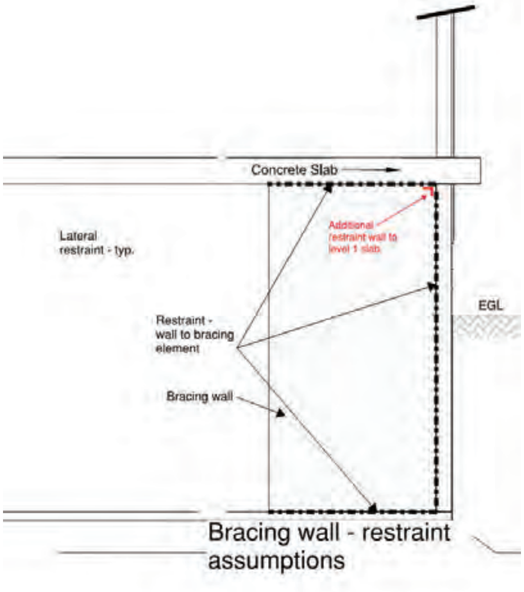


Figure 12 – bracing wall restraint assumptions

7.0 CONCLUSION

By Covey: In summary there are 3 sections of the rear retaining wall requiring attention.

I. Southern Stair Wall at grid 3:A/C also known as the *toilet area*.
II. Main Wall through grids 3:C/W
III. Northern Stair Wall from grid 3:W/Y also known as the *link area*.

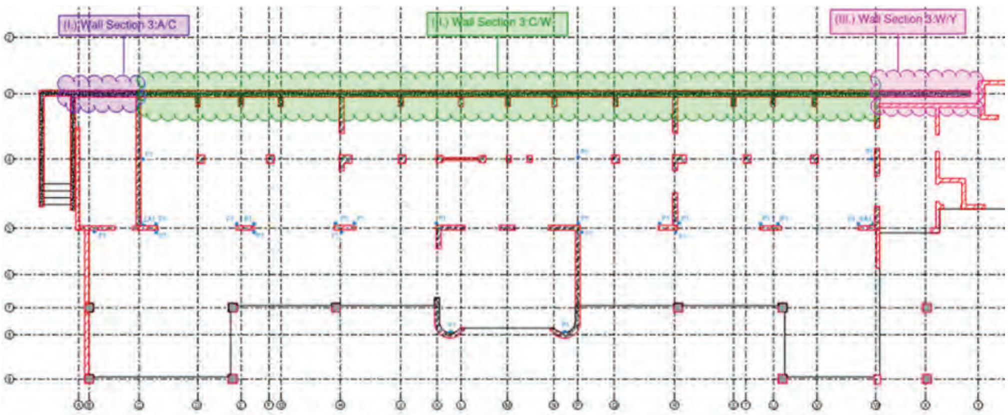


Figure 13: Retaining wall section references.

All sections of wall need strengthening works in addition to durability remedial works and waterproofing. If the drainage behind the wall was adequate to relieve the wall of hydrostatic pressure the associated strengthening works may be reduced.

7.1 Wall Section (I.) 3:A/C

Is apparently at the end of its service life (see BG&E Report).

Remedial works are more involved due to carbonation and chloride ingress.

A high level description of such works includes but is not limited to the following:

1. Ensure work area has full and complete access to all effect areas
2. Remove all carbonated and chloride affected concrete.
3. Ensure final thickness that will require reinstatement is less than 100mm.
4. Remove all surface rust from steel, wire brush clean surface. Ensure no section loss in the steel by using Callipers. If loss of section in reinforcement, contact engineer for reinforcement rectification works.
5. Apply protective coating on reinforcement.
6. Obtain specification, preparation, and installation instruction from suitably experienced and qualified Product manufacturer technical representative.
7. Use a bonding agent such as NITOBOND EP to be installed to manufacturers specifications.
8. Reinstate concrete under specialist advice.
9. Install strengthening requirements such as CFRP, tiebacks, buttresses etc.
10. Apply anti-carbonation coating.

7.2 Wall Section (II.) 3:C/W

This is the main wall through the internal section of the building which had carbonation to reinforcement, some chloride ingress, but not past the threshold (again see BG&E Report). At breakouts the reinforcement appeared to be in good condition with no corrosion.

Since the onset of corrosion of reinforcement has not yet occurred, there are more options for remedial intervention. Anti-carbonation coating is the most appropriate method and is expected to have a design life of up to 20 years (depending on the product manufacturers warranty etc.) alternatively sacrificial anodes can be used or replacement is also an option. (see BG&E report).
As already mentioned strengthening is required.

Nib/buttress supports need augmentation to provide enough strength.

7.3 Wall section (III.) 3:W/Y

Wall Section 3:W/Y (Old Northern stair rear wall or *Link area*) cannot be left as is and must either undergo remediation and strengthening or replacement.

Strengthening would require additional supports (and CFRP as mentioned above), but access is an issue as there is a block wall in front supporting the link roof framing etc and a very narrow gap between the walls. This limits the strengthening options to behind the wall. Unless the newer block wall is removed excavation behind the wall and support buttresses with new footings would be required.

Alternatively, if the newer block wall is opened up (partly demolished) or fully demolished for access then other options become available such as tiebacks/soil nails to embankment or internal buttresses.

Remedial works should be carried out in line with the Wall section 3:A/C and Beam CB1.

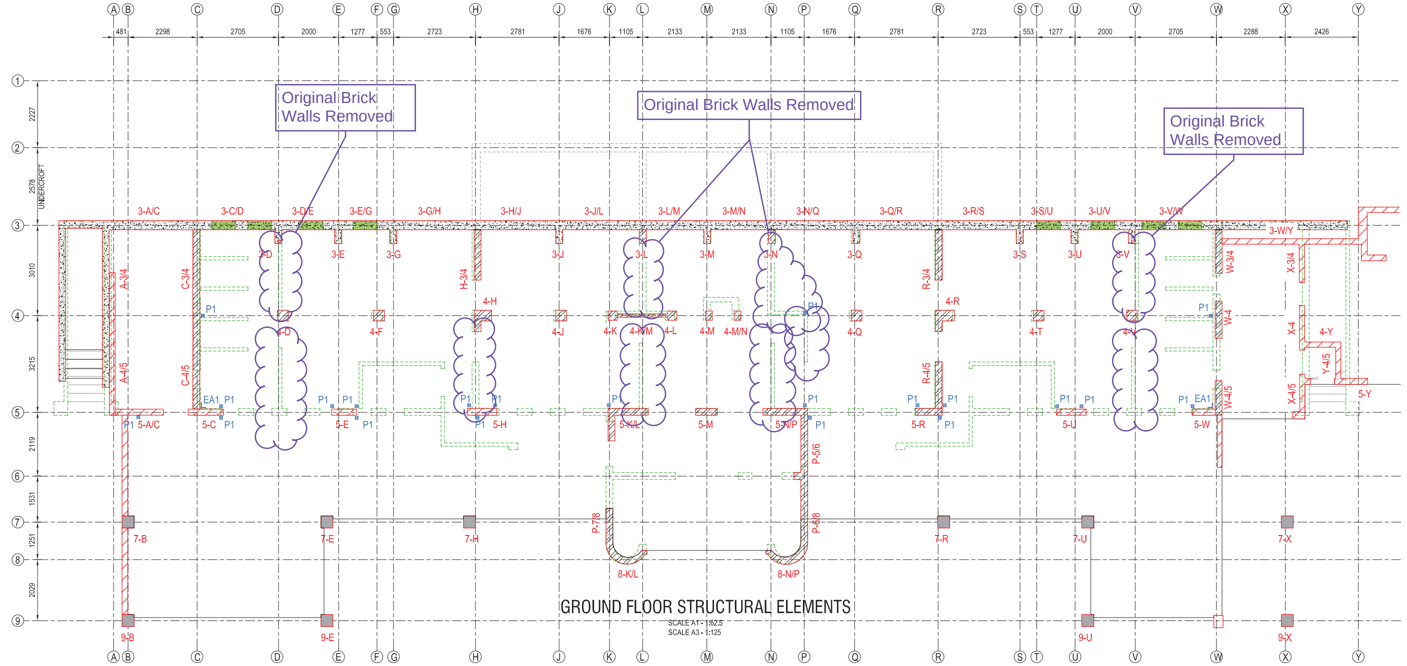
8.0 LIMITATIONS

The opinions, conclusions and any recommendations in this report are based on information from, and testing undertaken at or in connection with, specific sample points. Site conditions in other parts of the site may be different to those found at the specific sample points.

The opinions, conclusions and recommendations in this report are based on the assumptions made by Covey Associates and ACOR Consultants described in this report. Covey Associates and ACOR Consultants disclaim any liability arising from any of these assumptions being incorrect.

The opinions, conclusions and any recommendations in this report are based on the conditions encountered and information reviewed at the date of preparation of the report. Covey Associates and ACOR Consultants have no responsibility or obligation to update this report to account for events or changes subsequent to the date the report was prepared.

APPENDIX A



STRUCTURAL ELEMENT - IDENTIFICATION

ID	DESCRIPTION	MATERIAL	ORIGIN
3-AC	REAR WALL	IN-SITU CONCRETE	ORIGINAL
3-CD	REAR WALL	IN-SITU CONCRETE	ORIGINAL
3-DE	REAR WALL	IN-SITU CONCRETE	ORIGINAL
3-EG	REAR WALL	IN-SITU CONCRETE	ORIGINAL
3-GH	REAR WALL	IN-SITU CONCRETE	ORIGINAL
3-HJ	REAR WALL	IN-SITU CONCRETE	ORIGINAL
3-JL	REAR WALL	IN-SITU CONCRETE	ORIGINAL
3-LM	REAR WALL	IN-SITU CONCRETE	ORIGINAL
3-MN	REAR WALL	IN-SITU CONCRETE	ORIGINAL
3-NQ	REAR WALL	IN-SITU CONCRETE	ORIGINAL
3-QR	REAR WALL	IN-SITU CONCRETE	ORIGINAL
3-RS	REAR WALL	IN-SITU CONCRETE	ORIGINAL
3-SU	REAR WALL	IN-SITU CONCRETE	ORIGINAL
S-UV	REAR WALL	IN-SITU CONCRETE	ORIGINAL
3-VW	REAR WALL	IN-SITU CONCRETE	ORIGINAL
A-3/4	WALL	HALF CONCRETE/HALF BRICK	ORIGINAL /NEW
C-3/4	WALL	DOUBLE BRICK	ORIGINAL

STRUCTURAL ELEMENT - IDENTIFICATION

ID	DESCRIPTION	MATERIAL	ORIGIN
3-D	BUTTRESS	CONCRETE WITH BRICK CORNER	ORIGINAL
3-E	BUTTRESS	CONCRETE	ORIGINAL
3-G	BUTTRESS	CONCRETE	ORIGINAL
H-3/4	BUTTRESS WALL	DOUBLE BRICK	ORIGINAL
3-J	BUTTRESS	CONCRETE	ORIGINAL
3-L	BUTTRESS WALL	DOUBLE BRICK	ORIGINAL
3-M	BUTTRESS	CONCRETE	ORIGINAL
3-N	BUTTRESS WALL	DOUBLE BRICK	ORIGINAL
3-Q	BUTTRESS	CONCRETE	ORIGINAL
R-3/4	BUTTRESS WALL	DOUBLE BRICK	ORIGINAL
3-S	BUTTRESS	CONCRETE	ORIGINAL
3-U	BUTTRESS	CONCRETE	ORIGINAL
3-V	BUTTRESS	CONCRETE	ORIGINAL
W-3/4	WALL	DOUBLE BRICK	ORIGINAL
4-D	PIER	BRICK	ORIGINAL
4-F	PIER	BRICK	ORIGINAL
4-H	ENGAGED PIER	DOUBLE BRICK	ORIGINAL

STRUCTURAL ELEMENT - IDENTIFICATION

ID	DESCRIPTION	MATERIAL	ORIGIN
4-J	PIER	BRICK	ORIGINAL
4-K	PIER	BRICK	ORIGINAL
4-K/M	INFILL WALL	BRICK	ORIGINAL
4-L	PIER	BRICK	ORIGINAL
4-M	PIER	REMOVED	ORIGINAL
4-M/N	PIER	REMOVED	ORIGINAL
4-Q	PIER	BRICK	ORIGINAL
4-R	PIER	BRICK	ORIGINAL
4-T	PIER	BRICK	ORIGINAL
4-V	PIER	BRICK	ORIGINAL
4-W	WALL	DOUBLE BRICK	ORIGINAL
A-4/5	WALL	BLOCK	ORIGINAL
C-4/5	WALL	DOUBLE BRICK	ORIGINAL
R-4/5	WALL	DOUBLE BRICK	ORIGINAL
W-4/5	WALL	DOUBLE BRICK	ORIGINAL
5-A/C	WALL	BLOCK	2007
5-C	COLUMN	BLOCK	2007
5-E	COLUMN	BLOCK	2007
5-H	COLUMN	BLOCK	2007
5-K/L	WALL	DOUBLE BRICK	ORIGINAL

STRUCTURAL ELEMENT - IDENTIFICATION

ID	DESCRIPTION	MATERIAL	ORIGIN
5-M	WALL	DOUBLE BRICK	ORIGINAL
5-N/P	WALL	DOUBLE BRICK	ORIGINAL
5-R	COLUMN	BLOCK	2007
5-U	COLUMN	BLOCK	2007
5-W	COLUMN	BLOCK	2007
P-5/6	WALL	DOUBLE BRICK	ORIGINAL
7-B	COLUMN	CONCRETE	2007
7-E	COLUMN	CONCRETE	2007
7-H	COLUMN	CONCRETE	2007
P-7/8	WALL	DOUBLE BRICK	ORIGINAL
P-6/8	WALL	DOUBLE BRICK	ORIGINAL
7-R	COLUMN	CONCRETE	2007
7-U	COLUMN	CONCRETE	2007
7-X	COLUMN	CONCRETE	2007
8-K/L	WALL / PIER	BRICK	ORIGINAL
8-N/P	WALL / PIER	BRICK	ORIGINAL
9-B	COLUMN	CONCRETE	2007
9-E	COLUMN	CONCRETE	2007

STRUCTURAL ELEMENT - IDENTIFICATION

ID	DESCRIPTION	MATERIAL	ORIGIN
9-U	COLUMN	CONCRETE	2007
9-X	COLUMN	CONCRETE	2007
P1	100 SHS	STEEL	POST 2000
EA1	150x10 EA	STEEL	POST 2000
3-W/Y	WALL	BLOCK	2007
4-Y	WALL	BLOCK	2007
5-Y	WALL	BLOCK	2007
X-3/4	WALL	BLOCK	2007
X-4	WALL	BLOCK	2007
X-4/5	WALL	BLOCK	2007
Y-4/5	WALL	BLOCK	2007

LEGEND

ORIGINAL 1937 ARCHITECTURAL LAYOUT

CURRENT LAYOUT NOM. 100 mm +/-TOLERANCE

NOTE:
ALL DIMENSION ARE BASED OFF SCANNED PDF FROM 1937 ARCHITECTURAL SKETCHES, ROUGH SITE MEASURES. DIMENSIONS ARE APPROXIMATELY +/- 100mm
FOR MORE ACCURATE PLANS SEE DRAWINGS PRODUCED BY 3D CLOUD POINT.

Check all dimensions before commencement of work. Check Site boundary dimensions from the Title plans Check Building Boundary clearances by set out. Notwithstanding the information supplied on this drawing, the location, depth and extent of underground or overhead services are to be confirmed and protected on site by the contractor prior to commencement of works.

Checked – RG
Design – CJT Drawn – MS
Scales – AS SHOWN
Document Stage – INFORMATION

Issue	Date	Description	By
E	21.11.22	LAYOUT AMENDED	SAG
D	01.11.22	ISSUED FOR INFORMATION	TW
C	24.08.22	ISSUED FOR INFORMATION	RG
B	20.07.22	ISSUED FOR INFORMATION	CJT
A	15.07.2022	ISSUED FOR INFORMATION	MS

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Approved

Drawing title – PAVILLION 1 GROUND FLOOR STRUCTURAL ELEMENTS

Project – SUTTONS BEACH PAVILIONS

Client – MORETON BAY REGIONAL COUNCIL

Site – SUTTONS BEACH, REDCLIFFE, QLD, 4020

Project No. 223164 Sheet No. SK101
Digital Ref : 223164_SK101 GROUND FLOOR STRUCTURE Issue : E

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