

TESTING AND SEISMIC RETROFIT OF 1917 WINTEC F BLOCK URM BUILDING IN HAMILTON

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abSTRaCT

Wintec F block is a two storey unreinforced masonry (URM) building constructed in 1917 with an architectural style termed “stripped classical”, that was assessed to be potentially earthquake prone according to the provisions of the Building Act 2004. Material testing and seismic assessment were conducted on the as-built structure and it was determined that Wintec F block had sufficient out-of-plane seismic strength for most of its walls, but that the building had insufficient in-plane seismic strength. Seismic improvement was proposed using a type of strain-hardening fibre reinforced shotcrete called Engineered Cementitious Composite (ECC) combined with steel reinforcement, and the building was strengthened to 100% of New Building Standard (NBS).

1.0 INTRODUCTION

Unreinforced masonry (URM) buildings are a class of structure that were built using solid clay bricks and either lime or cement mortar, without the provision of supplementary reinforcement. Many URM buildings in New Zealand are potentially earthquake prone according to the provisions of the Building Act 2004 (Russell and Ingham 2010), which defines such buildings as having a seismic capacity level that is less than one third of that required for a new building (<33% of NBS) constructed at the same site. The seismic deficiency of URM buildings was again demonstrated recently in the 2010

Darfield earthquake (Dizhur et al. 2010a; Ingham and Griffith 2011), where many URM buildings exhibited in-plane diagonal tension failure (see Figure 1) or out-of-plane collapse of URM walls (see Figure 2).

The absence of tensile-resistant structural elements required to sustain earthquake loading, together with the frequent seismic activity that is expected due to New Zealand’s geographical location, necessitates seismic retrofit of many of New Zealand’s existing

URM buildings. Wintec F block is a two storey URM building in Hamilton that was assessed as a potentially earthquake prone building according to the provisions of the Building Act 2004 (NZ Parliament 2004).



Figure 1. In-plane diagonal tension failure of a URM building caused by the 2010 Darfield Earthquake

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Figure 2. URM building with collapsed walls due to out-of-plane acceleration in the 2010 Darfield Earthquake

Engineered Cementitious Composites (ECCs) are cement composites reinforced with synthetic fibres, which exhibit strain-hardening behaviour through the process of micro-cracking. When loaded in tension, cracks less than 100 μm develop and stress is transferred to the fibres that bridge the crack. As the fibres have a higher strength capacity than the cracking strength of the cement composites, multiple micro-cracks develop and strain-hardening behaviour is obtained. The strain-hardening behaviour of ECC makes it an ideal material for retrofitting of URM buildings, as it can provide both ductility and strength to the building. For retrofitting purposes, shotcreting is typically chosen as an application method because it is economical compared to casting of ECC (Kanda et al., 2003).

2.0 HISTORY AND CONSERVATION PLAN

2.1 History of Wintec F Block

In 1920, educators in New Zealand realized that in order to bolster economic growth, the education that they provided to the next generation of people needed to be focused on both trade and technical skills. With the ideals of turning farmers into semi-scientists (Sherson et al., 2006), the Hamilton Technical Day School was established. In 1924, boards and principal were appointed and the building became fully co-educational, with the school renamed as Hamilton Technical High School, with Whampoa Fraser being its first principal.

The Hamilton Technical High School was officially opened on 7th April, 1930 but at this time was already too small to accommodate the needs of its 400 students. Consequently, extensions to the Hamilton Technical High School were constructed along Anglesea Street and Nisbett Street in 1934 and 1948 respectively (Sherson et al., 2006), forming an "L" shape. The extensions were constructed of reinforced concrete instead of unreinforced

masonry, but the outside of the extension

had veneer bricks to maintain the architectural style. During 1945 the Hamilton Technical High School was renamed the "Hamilton Technical College".

In the 1960s, the residents of Hamilton urged the government to establish a tertiary education facility in south Auckland Province (Hamilton was part of Auckland Province at the time) and requested that this facility be located in Hamilton. Upon agreeing to Hamilton's request, the Waikato Technical Institute was formed and took over a section of the Hamilton Technical College's building, while the remaining section became Fraser High School (named after the first principal of the Hamilton Technical High School).

Two years later, Fraser High School was relocated to Ellicott Road and Waikato Technical Institute took over the remaining section of the building. Waikato Technical Institute is now known as Wintec, and the building it took over is now designated as F Block on the campus. Hence, the building is now known as Wintec F Block (see Figure 3) (Gibbons, 1977).



Figure 3. Wintec F Block

2.2 Architectural characteristic and conservation of Wintec F Block

Wintec F Block was designed by the government architect J. T. Mair and had an architectural style termed "stripped classical" (Sherson et al., 2006), which refers to two major features: The term "strip" refers to the absence of decorative components on the buildings, such as statues and ornaments; and the term "classical" refers to the use of classical columns and materials, featuring plain walls and vertical spandrels. From outside, the building has a recessed balcony and the building also has many distinctive internal features, such as terrazzo flooring, a curved staircase with iron balustrades and wood banisters which were all preserved in conjunction with the seismic retrofitting of the building.

The original building was constructed as a URM building with 3 to 4 leaf thick load bearing walls around the perimeter of the first level. On the second floor, all the walls decreased in thickness by one leaf with respect

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to the wall thickness directly below it. Two types of bricks were used in the building, with the original bricks dense and of a yellow color, and the other type of brick being dense and dark red, which are located primarily on the second level of the building and presumed to have been used when the building was extended in 1934 and 1948. On the outside of the building dark red veneer bricks were used to cover the exterior surface, while on the interior most walls are covered with plaster to a thickness ranging between 10 to 15 mm. Roof and floor diaphragms inside the building are constructed of Matai timber and floor joists on the first level have the dimensions of 190 mm × 45 mm, with a spacing of 450 mm c/c. Multiple joists had previously become damaged by moisture and were found to be cracked and broken in several locations. On the second level, the floor diaphragms consist of timber joists with dimensions of 450 mm × 45 mm at the same 450 mm c/c spacing. The concrete extensions constructed in 1934 and 1948 are located at both ends of the "L" shape building (see Figure 4).

3.0 TESTING OF AS-BUILT STRUCTURAL ELEMENTS

3.1 Material properties

The constituent material properties of Wintec F Block were determined through irregular mortar samples,

single bricks and three brick high masonry prisms extracted from the building site and tested in the laboratory. The masonry consisted of bricks with dimensions of

75 mm × 220 mm × 105 mm and mortar joints being 10 mm thick on average. Examples of the masonry type observed are shown in Figure 5.

3.1.1 Brick compression strength

The brick compression strength was obtained using the half brick compression test method ASTM C67-00 (ASTM 2001c), with results presented in Table 1 where f'_b is the half brick compressive strength, and the coefficient of variation is reported in parenthesis. The bricks were yellow in colour and were classified as soft bricks based on their physical appearance according to the NZSEE (2006) guideline. The NZSEE 2006 recommended strength range for soft bricks is also shown in Table 1. The actual compressive strength range shown in Table 1 significantly exceeded the NZSEE recommendation for the corresponding brick type.

3.1.2 Mortar compression strength

Mortar samples were extracted from the exterior wall of the building to investigate mortar properties. The mortar from the building appeared to be cement-lime mortar

Table 1. Half brick compressive test results and the NZSEE (2006) suggested range

Sample Size	f'_b (MPa)
5	25.9 (0.25)
Range (MPa)	
20.1	
35.8	
NZSEE Range (MPa)	
-	
1-5	

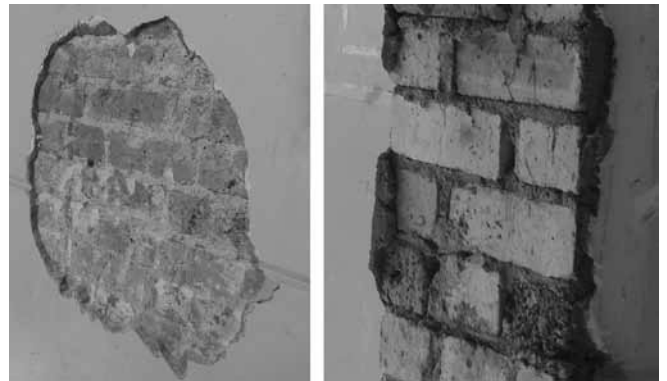


Figure 5. Masonry type observed in Wintec F Block

Table 2. Mortar testing results

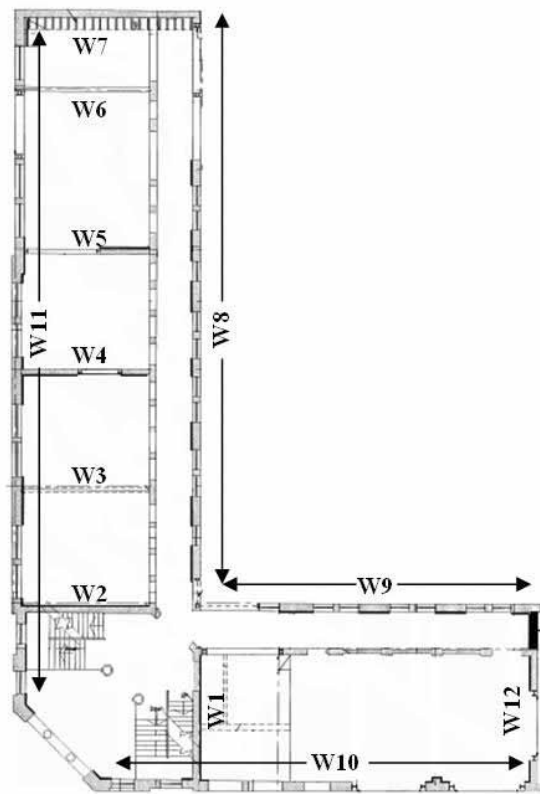


Figure 4. Building plan for Wintec F Block

Sample		
size	f_j (MPa)	Range (MPa)
6	17.68 (0.46)	8.0 – 26.6

Table 3. Average masonry prism compressive strength (f_m) and Young's modulus (E)

	Average	CoV	Sample Size
f_m (MPa)	9.7	0.18	5
Range	7.6 – 11.4		
E (GPa)	2.5	0.59	
E ($\times f_m$)	268	0.27	



Figure 6. Mortar sample in testing

and in good condition. In internationally established standards such as ASTM C 109M-99 (ASTM 2001a), compression testing of mortar cylinders or 50 mm mortar cubes is recommended. Extraction of samples in such size and shape is unattainable in existing buildings as the mortar joint thickness is limited, and therefore irregular mortar samples were acquired, cut into measurable shape (36 mm long × 24 mm wide × 30 mm high) and tested in compression. Two extra samples having dimensions of 20 mm long × 20 mm wide × 30 mm high were also tested, and these results were found to be low compared to those of the larger samples. The results of mortar testing are shown in Table 2 and the test is shown in Figure 6. The high coefficient of variation was due to the low compressive strength of the smaller samples. The average mortar compressive strength was higher than the NZSEE recommendation for stiff mortar (8.0 MPa) (NZSEE, 2006).

3.1.3 Masonry prism compressive strength and Young's Modulus

A number of three brick high single leaf masonry prisms were extracted from the interior wall W4 (see Figure 4) and tested in compression in accordance with ASTM 1314-00a (ASTM 2001b). Displacement gauges were incorporated during testing to obtain the stress-strain response and Young's Modulus values (E) as presented in Table 3. Figure 7 shows a masonry prism during testing. The average masonry compressive strength was 9.7 MPa, which is higher than the FEMA

356 recommendation for masonry in good condition (6.2 MPa). However the average masonry Young's modulus (2.6 GPa) was lower than the FEMA suggested value of 3.4 GPa (FEMA, 2000). Comparison with the NZSEE (2006) guideline could not be made as the guideline does not provide the recommended values for masonry prisms.



Figure 7. Masonry prism during testing

3.2 Out-of-plane testing

In the event of an earthquake, eight out-of-plane failure modes for URM walls have been identified (D'Ayala and Speranza 2003). The study reported here focused on the failure mode that involves the development of horizontal cracks at the wall mid-height, followed by wall rocking and ultimately spalling of the wall when the critical displacement is reached (Derekhshan et al. 2011; Griffith et al., 2004). The targeted failure mode generally occurs in URM walls that are non load-bearing and simply supported vertically.

Two out-of-plane URM wall tests were conducted on a non-load bearing, 2-leaf clay brick masonry partition wall (W3 in Figure 4). The wall had a 15-20 mm thick cement plaster finish on both sides, and had dimensions of 7.2 m long \times 3.9 m high (see Figure 8). In order to simplify the tests and to provide direct comparison to the results obtained from laboratory-built specimens and the assessment results based on NZSEE (2006), a 1250 mm wide strip of wall was isolated from the rest of the wall, thus inducing a one way bending failure. The location of the strip was selected adjacent to a door opening to minimise the length of required saw cuts as shown in Figure 9.

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Due to the presence of a concrete ring beam at the top of the wall, the as-built testing consisted of two schemes to investigate the effects of different boundary conditions. The first scheme (Test AS-CB) involved cutting the wall strip up to the concrete ring beam and testing with the continuous concrete beam extended across the wall strip. The second scheme (Test AS-SS) involved cutting the concrete ring beam, including all reinforcement, and providing an additional horizontal support at the top of the wall strip (connected to the first floor joists), resulting in a propped support at the top (see Figure 10). The experimental setup (see Figure 11) closely resembled that which Derakhshan et al. (2009) used to conduct laboratory out-of-plane testing on URM walls at the University of Auckland. Two Bigfoot vinyl airbags, measuring 2.1 m \times 1.2 m, were inflated to gradually apply the loading. Due to a reduction in the actual inflated airbag area resulting from the rounded geometry of the airbag edges, the recorded applied force was observed to be between 80% to 85% of that calculated using airbag pressure and gross loaded area (deflated airbag area). The plywood backing, measuring 2.4 m \times 1.2 m, consisted of an assemblage of timber sheets and steel angles, and was supported by a reaction frame. As the diaphragm was incapable of resisting the generated forces, timber planks were used underneath the setup frames (Figure 11) to distribute the reaction force on the diaphragm surface and to partially transfer the reaction forces to the opposite wall foundation.

The reaction forces were transferred to the reaction frame using four load cells, and to ensure that the entire force was transferred through the load measuring devices, frictionless plates were used underneath the plywood backing. Out-of-plane displacement was measured using two Linear Variable Differential Transducers (LVDT) mounted on the opposite side of the wall, with one LVDT placed mid-height and one at

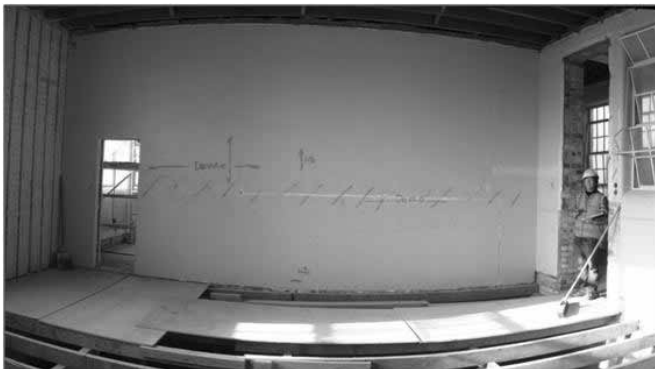


Figure 8. Panorama (distorted) view of Wall W3

the wall centerline. The data from load cells and LVDTs were collected at 50 Hz frequency using a National Instruments data acquisition system.

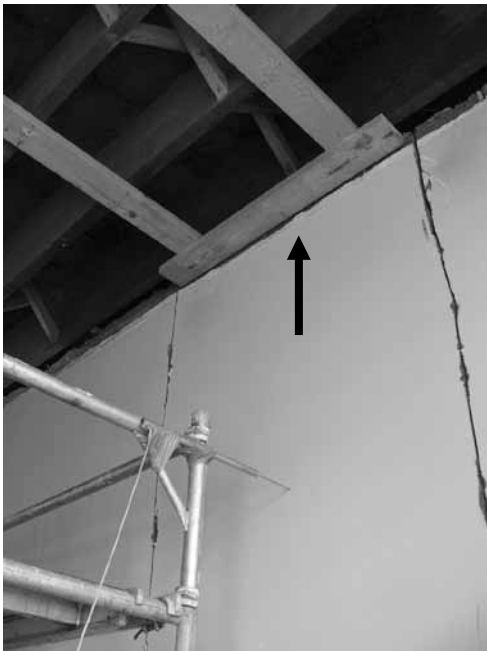


Figure 9. Cutting the wall strip using a masonry-cutting chainsaw

Timber support

Figure 10. Timber support constructed for AS-SS test



Figure 11. Reaction frame constructed for out-of-plane testing

During Test AS-CB no cracking was visually detected on the wall surface, although the lateral force was increased up to approximately 24 kN. Consequently, the obtained results for test AS-CB were all in the elastic range (see Figure 12).

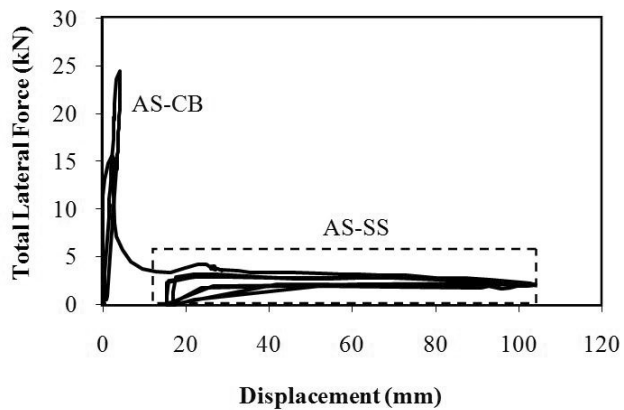


Figure 12. Force-displacement graph

Testing of AS-SS resulted in a single horizontal crack at 56% of the wall strip height (see Figure 13) above the base, with the crack location being consistent with the laboratory testing previously conducted by Derakhshan et al. (2009). The test was continued, and in three semi-cycles of loading, a maximum lateral wall mid-height displacement of 105 mm was measured. The resulting force-displacement history of the three semi-cycle loading (shown in the dashed box in Figure 12) is isolated and shown in Figure 14, showing good correlation between the experimental results and a bilinear model previously obtained following laboratory tests reported by Derakhshan et al. (2011). The wall strip had a 19 mm residual displacement after the completion of test AS-SS. Full details of the testing can be found in Dizhur et al. (2010b).



Figure 13. Wall crack during the AS-SS test

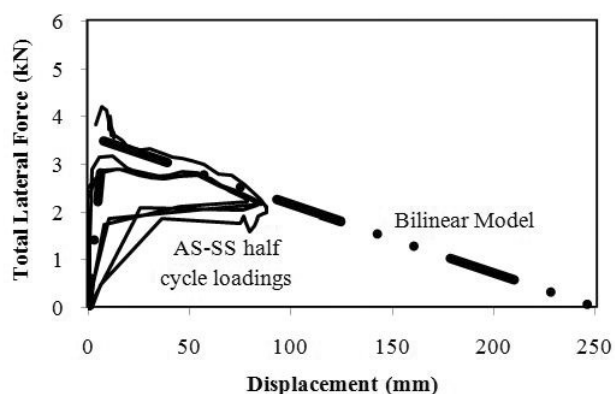


Figure 14. Wall post-cracking behavior

4.2 Out-of-plane assessment

To establish out-of-plane wall stability, URM walls were assumed to be non-load bearing and act in a one-way bending condition. The assessment method used herein assumed that the wall had no strength in the out-of-plane direction and that when loaded, stable rocking of the wall occurs. As most walls have a concrete ring beam as top support, the evaluation method and assumptions made provide a conservative estimate of wall strength. Wall height-to-thickness ratio (h/t), out-of-plane displacement capacity, out-of-plane displacement demand and %NBS related to the displacement were calculated according to Ingham (2011) and are presented in Table 6 and 7. An equivalent moment demand related to the displacement demand is also incorporated in Table 6 and 7 as the strengthening procedure is force based as opposed to displacement based.

1.1	191.6	246.7	77.7
2.1	191.6	196.9	97.3
3.1	Wall demolished		
4.1	74.8	123.1	60.8
5.1	33.3	82.1	40.5
6.1	Concrete wall		
7.1	Concrete wall		
8.1	16.0/pier	61.0	26.3
9.1	16.0/pier	49.2	32.6
10.1	8.0/pier	34.7	23.0
11.1	49.9	107.6	46.4
12.1	Concrete wall		

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Table 6. Out-of-plane assessment of level 2

Wall	h/t ratio	Displacement capacity (mm)	Displacement demand (mm)	Equivalent moment demand (kNm)	%NBS
1.2 & 2.2	22	132	70	7.3	188
3.2	Does not exist				
4.2	20	132	96	7.0	138
5.2	Does not exist				
6.2	Does not exist				
7.2	Concrete wall				
8.2 & 9.2	14	198	79	2.5	250
10.2	14	198	79	1.8	250
11.2	14	198	79	4.4	250

Table 7. Out-of-plane assessment of level 1

Wall	h/t ratio				
Displacement capacity (mm)					
Displacement demand (mm)					
Equivalent moment demand (kNm)	%NBS				
1.1 & 2.1	14	198	244	7.1	81
3.1	Wall demolished				
4.1	14	198	236	4.4	84
5.1	14	198	244	2.9	81
6.1	Concrete wall				
7.1	Concrete wall				
8.1 & 9.1	10	264	300	1.5	88
10.1	10	264	300	1.1	88
11.1	10	264	300	2.7	88
12.1	Concrete wall				

5.0 DESIGN OF SEISMIC RETROFIT

The seismic retrofit design reported here aimed to strengthen the building to 100% of new building strength and the design approach proposed by Ingham (2011) was adopted for all design procedures. The ECC design properties were based on the following values,

was assessed using Equation 1, where V_{fd} is the shear strength contribution of ECC, t_{frs} and f'_{frs} are the thickness and tensile strength of ECC respectively, and z is the effective length, taken as 0.72 of the wall length being strengthened.

as provided by the ECC shotcrete supplier:

ଫିଲ୍ଡ୍ ଚିଂଟ୍ କଣ୍କ୍ରିଟ୍

(1)

$$\begin{aligned}
 f'_{ECC} &= 3.0 \text{ MPa} && \text{(tensile strength)} \\
 f_{ECC} &= 40 \text{ MPa} && \text{(compressive strength)} \\
 E_{ECC} &= 16 \text{ GPa} && \text{(elastic modulus)} \\
 \varepsilon_{yECC} &= 0.0125\% && \text{(yield strain)} \\
 \varepsilon_{uECC} &= 1\% && \text{(maximum strain)}
 \end{aligned}$$

For walls where ECC could be applied on both surfaces, in-plane strengthening design was conducted first and the wall was then assessed for out-of-plane strength. For walls where ECC could only be applied on a single surface, the out-of-plane design with ECC acting on the compression surface of loading was conducted first, followed by in-plane assessment.

5.1 In-plane strengthening philosophy

After confirming the ECC section had sufficient shear strength to sustain the seismic demand, the bond area was checked to ensure it was sufficient to transfer the shear force from the masonry to the ECC reinforcement and avoid a bond failure. Equation 2 was used to check the shear debonding strength, where k_{wt} is a factor considering the effectiveness of external bonded reinforcement as wall thickness increases, which can be taken as 1.0 for 2-leaf walls or walls with ECC on both surface. f'_b is the brick compression strength and E_{frs} is the stiffness of ECC, both in

3.2			Wall demolished		
4.2	2	20	N/A	304.6	742.2
5.2			Does not exist		
6.2			Concrete wall		
7.2			Concrete wall		
8.2	1	20	850	213.1	123.0
9.2	1	20	850	223.6	123.0
10.2	1	20	600	215.2	174.3
11.2	1	20	1000	233.8	104.6
12.2			Concrete Wall		

Table 9. Location of ECC reinforcement on level 1

Wall	Surfaces reinforced				
ECC thickness/ surface (mm)					
Steel reinforcement spacing (mm)	In-plane %NBS	Out-of-plane %NBS			
1.1	2	20	N/A	172.2	1040.8
2.1	2	20	N/A	215.7	1040.8
3.1			Wall demolished		
4.1	2	20	N/A	179.3	1040.8
5.1	2	20	N/A	159.0	1040.8
6.1			Concrete wall		
7.1			Concrete wall		
8.1	1	20	850	116.6	201.0
9.1	1	20	850	144.5	201.0
10.1	1	20	600	134.9	284.7
11.1	1	20	1000	136.7	170.8
12.1			Concrete Wall		

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6.0 CONCLUSIONS

Material properties and site testing were conducted on Wintec F block, and the information obtained from testing of the building was used to assess its seismic performance. The results of the assessment showed that the building had only 20% of the NBS requirement and that a seismic retrofit of the building was required. Using a combination of ECC and steel reinforcement, and following the procedures reported in Ingham (2011), the new %NBS of Wintec F Block was increased beyond 100% NBS.

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APPENDIX: IMPLEMENTATION PHOTOS OF WINTEC F BLOCK

Figure 16. Water-blasting of masonry surface to prepare for shotcrete



Figure 17. NSM steel reinforcing bars



Figure 18. Shotcreting of W4



Figure 19. W1 after shotcreting