HOLMESSOLUTIONS

REPORT 107267-1 (V1.1)

PREPARED FOR ROYAL COMMISSION

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MATERIALS TESTING IN BUILDINGS OF INTEREST

GALLERY APARTMENTS
WESTPAC CENTRE
IRD BUILDING

HOLMESSOLUTIONS

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This document does not constitute a standard, specification, or regulation. In undertaking the testing described in this report, Holmes Solutions have exercised the degree of skill, care, and diligence normally expected of a competent testing agency. The name of specific products or manufacturers listed herein does not imply endorsement of those products or manufacturers.

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1.0 EXECUTIVE SUMMARY

Following the recent Christchurch earthquakes significant structural damage was noted in a large number of buildings in the Christchurch CBD. In particular, a number of buildings appear to have undergone greater damage than previously expected. The Royal Commission appointed an engineering team to review the damage in a number of building in the CBD in an effort to gain a greater understanding of the buildings behaviour under the induced seismic loads. From this investigation, a series of three buildings were identified as requiring materials testing to be completed, namely the Gallery Apartments on Glouster St, the Westpac Centre on Cashel St, and the IRD building on Cashel St. Holmes Solutions was commissioned to undertake the required materials testing.

All three buildings requiring investigation are reinforced concrete, with a mixture of precast concrete and in-situ cast concrete elements. The Royal Commission requested a series of destructive and non-destructive testing to be completed on the concrete and reinforcing steel used in the buildings. Furthermore, Holmes Solutions was independently engaged by external third parties working for the owners of the building to undertake additional testing on the reinforcing steel in the Westpac Centre and IRD building.

Testing of the concrete elements included the removal of concrete cores for destructive testing to determine the tensile and compressive properties of the concrete. Additional non-destructive testing was completed using Schmidt Hammer testing in the buildings.

The material properties of the reinforcing steel were investigated in zones of damage in the building, to determine the likely damage the earthquake has induced in the steel, and control samples in areas away from any noted damage. The use of Leeb Hardness testing has been shown to provide a strong correlation with the peak strain the steel has been subjected to during in-elastic loading cycles and is become increasingly adopted as a tool for assessing structural damage.

The results from the testing indicated that the reinforcing steel in the Westpac Centre had undergone previous inelastic strain cycles of between 2% and 8%. The reinforcing steel testing in the IRD building showed significant reduction in strain capacity with only 2% strain capacity remaining.

Concrete strength results for the Gallery Apartments indicated that the walls had compressive strengths of 46 MPa to 56 MPa, with associated tensile strengths ranges from 3.4 MPa to 2.6 MPa respectively.

No significant variations in concrete strengths were noted between the precast and in-situ concrete items in the Westpac Centre.

Concrete results from the IRD indicated that the precast concrete was stronger than the in-situ concrete elements by approximately 10 MPa.

2.0 TEST METHODOLOGY

2.1. CONCRETE CORE TESTING

A series of concrete core samples, approximately 100 mm in diameter, were removed from elements in the Gallery Apartment and the Westpac Centre. The cores were removed using a diamond tipped drilling head. Wherever possible, samples were taken from areas showing no physical damage and remote from reinforcing steel embedded in the concrete. If a reinforcing bar was impacted by the drilling head, the sample was discarded and an alternative sample taken from a nearby position. Prior to removing the core, the orientation of the sample was clearly identified to allow the subsequent testing to be undertaken in the correct orientation.

The concrete cores were subjected to either tensile splitting tests or compression testing. All tensile splitting tests were performed to the specific requirements of NZS 3112: 1986, Pt 2, Clause 8. Care was taken to ensure the samples were oriented as per location in the building. All samples were prepared in accordance with the standard prior to completion of the testing.

All concrete cores subjected to compression testing were firstly capped, in accordance with the requirement of NZS 3112: Part 2: 1986, clause 4. Once the capping material had achieved the required hardness the samples were tested in accordance to NZS 3112: Part 2: 1986, Clause 6.

2.2. TENSILE STEEL TESTS

A series of steel samples, approximately 500 mm long were removed from the Westpac Centre and the IRD building. Steel samples from the Westpac centre were obtained from zones of noted damage in the building and additional samples collected from areas that appeared to be free of visual damage to act as control samples and provide a true measure of the stress-strain properties of the parent steel. Prior to their removal from the Westpac Centre, all steel bars were subjected to Leeb Hardness testing in-situ.

2.3. LEEB HARDNESS

Leeb hardness is a direct measure of a materials dynamic hardness and is considered to be accurately measuring the materials elastic and plastic hardness characteristics. Leeb hardness is obtained by firing an impact body containing a permanent magnet and a very hard indenter sphere towards the surface of the test material and measuring the velocity of the impact body. The velocity is measured in three main test phases;

- Pre-impact phase, where the impact body is accelerated by spring force towards the surface of the test piece.
- Impact phase, where the impact body and the test piece are in contact. The hard indenter tip deforms the test material elastically and plastically and is deformed itself elastically. After the impact body is fully stopped, elastic recovery of the test material and the impact body takes place and causes the rebound of the impact body.
- Rebound phase, where the impact body leaves the test piece with residual energy, not consumed during the impact phase.

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The Leeb hardness is determined by calculation, relating the three recorded velocities. The velocities are measured in a contact-free means via the induction voltage generated by the moving magnet through a defined induction coil mounted on the guide tube of the device. The induced voltage is directly proportional to the velocity of the magnet and therefore used to determine the hardness of the steel sample.

Recent research has shown that hardness can be used as an indicator of the current strain state of steel samples [G1, L1, M2, N2, N3]. Relating the hardness of steel samples to the stress-strain properties of the base material allows an understanding of likely damage (or loss of strain capacity) that the steel sample has sustained and therefore to determine how much residual strain capacity the sample retains. This form of direct comparison can only be achieved if suitable correlations are developed between the measured hardness and the strain state of the specific steel sample.

Holmes Solutions has completed extensive research into the correlation between Leeb hardness and the steel samples strain state for a range of different reinforcing steels. The results from the research have been developed into a series of multi-dimensional correlation factors. When combined with a series of normalisation techniques we can use the measured Leeb hardness results to provide an indication as to the current strain state of the tested steel sample. The degree of uncertainty in the recorded measurements is decreased through the physical testing of a control section of the steel to a uniaxial tension test and undertaking hardness measurements at a series of predefined stress and strains. The resulting correlation is used, in conjunction with the normalisation techniques derived from obtaining numerous hardness readings in the area surrounding the expected zone of damage, to determine the value of strain in the steel from the recorded Leeb measurements. These results are then directly compared to the properties of the parent material to estimate the potential reduction in strain capacity that has been sustained by the steel sample.

Leeb readings are collected from in-situ reinforcing bars. The surface of the bars is carefully prepared to specific requirements prior to testing. Readings are obtained at critical locations along the length of the reinforcing bar to allow the strain profile of the steel to be determined and to assist in the normalisation procedures.

The overall estimation of strain degradation for the tested steel samples is achieved by using the derived strain damage from the Leeb testing in conjunction with engineering knowledge of the particular application.

All in situ hardness testing is completed in accordance with ASTM A959-06 Standard Test Methods for Leeb Hardness Testing of Steel Products [A2]. For all locations, a minimum of 6 individual hardness tests were completed with the results averaged to obtain the recorded Leeb value [A1]. All recorded values were then normalised using the derived multi-dimensional correlation factors.

2.4. CONCRETE REBOUND HARDNESS

Concrete hardness is often used as a non-destructive means of determining the compressive strength of concrete. The most common method employed is the rebound hardness, obtained from a portable Schmidt Hammer. The Schmidt hammer works using a similar principle to the Leeb Hardness measurements, whereby a weight is impacted on the surface of the material and the change in velocity between the impact speed and rebound speed is determined. Correlations are then applied to convert the change in speed to hardness and compressive strength.

As with the Leeb Hardness measurements, increased accuracy in the obtained results is achieved if the hardness measurements can be directly correlated against the specific material being tested, by completing destructive materials testing on samples of the material. This s typically achieved by removing core samples from the structure and subjecting them to compressive testing. However, if no materials testing is completed, standard conversion tables can be used to form the correlations, with an associated reduction in accuracy.

The correlations for the Gallery Apartments and Westpac Centre were completed using the results from the physical testing of concrete core samples removed the buildings. No cores could be removed from the IRD building and as such the standard lower 10 percentile strength curves specifically developed for the instrument used in the testing. The curves were derived from testing of over 2,300 discrete locations. Use of the lower 10 percentile curve is recommended by the leading Standards, EN 13791 and ASTM C805/ACI 228.1.

In each tested location, a grid of readings were recorded. The results from the grid of readings were then averaged to provide the concrete hardness and associated concrete strength of that location. This testing method is endorsed by most International Testing Standards, and the manufacturers of the test equipment.

Steel samples from the IRD building were supplied to HSL by the engineers who designed the building. The steel samples were taken from a damaged zone in the central core of the building. Leeb Hardness testing was completed on the steel samples prior to the completion of the physical tensile testing.

All tensile testing was completed to the requirements of ASTM E8/ E8M:08.

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3.0 TEST EQUIPMENT

3.1. LEEB HARDNESS TESTER

A Proceq Equotip 3 portable hardness tester was used to collect all material hardness values. The device is generally acknowledged as the industry standard for the determination of Leeb hardness. The hardness tester was installed with a DL impact device, allowing measurements on smaller diameter steel samples than the conventional D device.

The Equotip 3 has a reported accuracy of ±4 HL and is traceably calibrated to NIST standards.

3.2. SILVERSCHMIDT HAMMER

A Proceq Silverschmidt Rebound Hammer was used to undertake all field based concrete hardness testing for concretes of compressive strength ranging from 10 to 100 MPa. This device and methodology generally accepted as the industry leading device for determining the compressive strength of concrete in-situ.

The Proceq Silverschmidt was fitted with the N-Type rebound hammer providing test impact energy of 2.207 Nm.

3.3. UNIVERSAL TEST MACHINE

A UH600 Shimazu servo-controlled Universal Test Machine (UTM) with a 600 kN capacity was used to undertake all laboratory based materials testing. The UTM has a maximum stroke of 250 mm and a peak table velocity of 150 mm/min.

Steel Elongation was recorded using a strain gauge based digital extensometer with a gauge length of 50 mm. Applied loads were recorded directly using the internal pressure transducer of the Shimazu control system.

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4.0 GALLERY APARTMENT RESULTS

4.1. CONCRETE RESULTS

A series of four concrete cores were removed from the concrete shear wall elements towards the front of the Gallery apartments. Two cores were subjected to uniaxial compression testing whilst the remaining two cores were subjected to split cylinder testing in order to determine the tensile properties of the concrete. The results from the physical testing on the cores are presented below.

Table 1 Compressive Cylinder results for the Gallery Apartment

Specimen Name		**RWRC**	**FWRC**
Date Tested		10 Nov 2011	10 Nov 2011
Age	(days)	Unknown	Unknown
Size & Position of any reinforcing		None	None
Visual description		Homogeneous	Homogeneous
Average core diameter	(mm)	94.1	93.9
Average core length (upon receipt)	(mm)	255.6	254.8
Average core length (after docking)	(mm)	190.0	187.6
Mass of core prior to capping	(g)	3191	3098
Density	(kg/m³)	2421	2387
Height diameter ratio		2.02	2.0
Conditioning		Air dried	Air dried
Load at Failure	(kN)	388.8	322.1
Compressive Strength	**(MPa)**	**56.0**	**46.5**
Type of fracture		column	Shear

Table 2 Split Cylinder results for the Gallery Apartment

Specimen Name		**RWLC**	**FWLC**
Date Tested		11 Nov 2011	11 Nov 2011
Age	(days)	Unknown	Unknown
Defects in cylinder		None	None
Visual description		Homogeneous	Homogeneous
Average core diameter	(mm)	93.6	94.0
Average length	(mm)	189.5	167.5
Mass of cylinder in air	(g)	3133	2742
Density	(kg/m³)	2400	2380
Height diameter ratio		2.02	1.78
Conditioning		Air dried	Air dried
Tensile Strength	**(MPa)**	**2.4**	**3.4**

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In addition to physical testing, a series of Schmidt hammer tests were completed in additional locations surrounding the noted zones of damage in the building. The results from the Schmidt hammer tests are presented below.

The conversion from hardness information into concrete cylinder compressive strength is presented utilises the standard conversion factors typically use with Schmidt hammers, which has been derived from extensive testing on concrete samples in Europe. The results indicate that the normalised correlation curves typically overestimated the actual concrete strength when compared to the actual concrete strength information obtained from the concrete cores that were tested.

Table 3 Schmidt Hammer test results for Gallery Apartments

location: Front Wall - Left Side

	1	2	3	4
A	73	71.5	72	
B	67	73.5	77	72
C	72.5	72.5	71.5	70
D		60	70.5	72

Correct Average:	71.8	
Cube Strength:	87.1	MPa
Cylinder Strength, f'c:	**70.0**	**MPa**

location: Front Wall - Right Side

	1	2	3	4
A	68.5	70.5	67.5	
B	73.5	71	71	71.5
C	72	75.5	66.5	70.5
D		72	70.5	73.5

Correct Average:	70.8	
Cube Strength:	82.8	MPa
Cylinder Strength, f'c:	**66.0**	**MPa**

location: Rear Wall - Left Side

	1	2	3	4
A	75	73	62.5	
B	65	70.5	75	68.5
C	67.5	67.5	69.5	62.5
D		68.5	70.5	68.5

Correct Average:	70.0	
Cube Strength:	80.2	MPa
Cylinder Strength, f'c:	**63.0**	**MPa**

location: Rear Wall - Right Side

	1	2	3	4
A	73	65	72	
B	69.5	65	65	61
C	69.5	64	65	64.5
D		58.5	74	63

Correct Average:	66.2	
Cube Strength:	67.4	MPa
Cylinder Strength, f'c:	**54.0**	**MPa**

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Figure 1 *Drilling concrete core from Gallery Apartments*

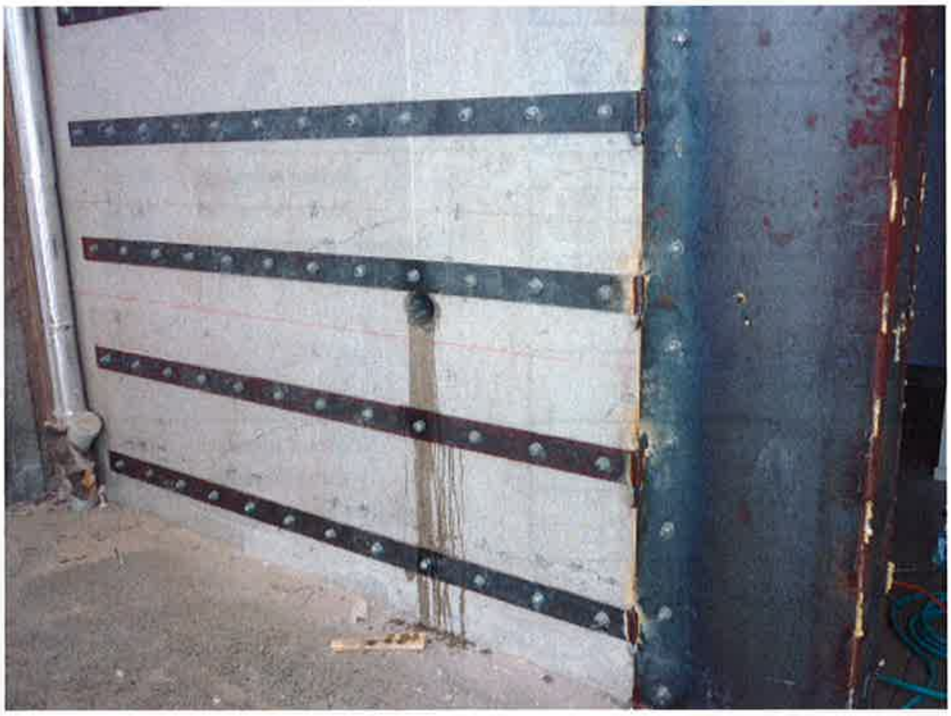

Figure 2 Core removed from Gallery Apartment Wall

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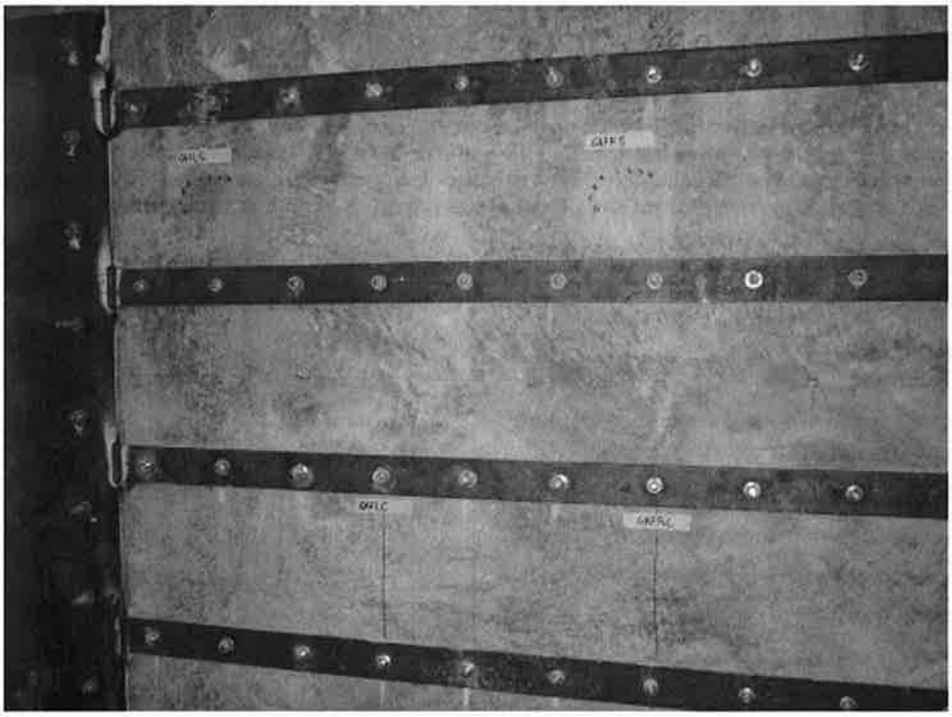

Figure 3 *Test locations on Front Wall of Gallery Appartments*

Figure 4 Schmidt Hammer test location GAFLS

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5.0 WESTPAC CENTRE RESULTS

5.1. CONCRETE RESULTS

A series of 6 concrete cores were removed from the concrete elements, all of which were subjected to compression testing. Two of the cores were removed from precast beams, two from column elements, and the remaining two were extracted from the in-situ walls. The results from the physical testing on the cores are presented below.

Table 4 Compressive Cylinder results for the Precast beams in Westpac Centre

Specimen Name		**Precast Beam 2**	**Precast Beam 3**
Date Tested		10 Nov 2011	10 Nov 2011
Age	(days)	Unknown	Unknown
Size & Position of any reinforcing		None	None
Visual description		Homogeneous	Homogeneous
Average core diameter	(mm)	93.8	93.9
Average core length (upon receipt)	(mm)	227.3	211.0
Average core length (after docking)	(mm)	192.0	188.1
Mass of core prior to capping	(g)	3032	2920
Density	(kg/m^3)	2311	2253
Height diameter ratio		2.05	2.00
Conditioning		Air dried	Air dried
Load at Failure	(kN)	158.4	149.5
Compressive Strength	**(MPa)**	**23.0**	**21.5**
Type of fracture		shear	shear

Table 5 Compressive Cylinder results for the In-situ walls in Westpac Centre

Specimen Name		**In-situ wall - Bottom**	**In-situ wall - Top**
Date Tested		10 Nov 2011	10 Nov 2011
Age	(days)	Unknown	Unknown
Size & Position of any reinforcing		None	None
Visual description		Homogeneous	Homogeneous
Average core diameter	(mm)	93.7	94.1
Average core length (upon receipt)	(mm)	234.5	218.5
Average core length (after docking)	(mm)	191.1	193.1
Mass of core prior to capping	(g)	3028	3068
Density	(kg/m^3)	2315	2305
Height diameter ratio		2.04	2.05
Conditioning		Air dried	Air dried
Load at Failure	(kN)	134.5	119.2
Compressive Strength	**(MPa)**	**19.5**	**17.0**
Type of fracture		column	shear

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Table 6 Compressive Cylinder results for the Circular columns in Westpac Centre

Specimen Name		**Column 1**	**Column 2**
Date Tested		10 Nov 2011	10 Nov 2011
Age	(days)	Unknown	Unknown
Size & Position of any reinforcing		None	None
Visual description		Homogeneous	Homogeneous
Average core diameter	(mm)	94.1	94.2
Average core length (upon receipt)	(mm)	223.1	154.8
Average core length (after docking)	(mm)	185	123
Mass of core prior to capping	(g)	3074	1992
Density	(kg/m³)	2394	2344
Height diameter ratio		1.97	1.31
Conditioning		Air dried	Air dried
Load at Failure	(kN)	158.4	224.2
Compressive Strength	**(MPa)**	**23.0**	**32.0**
Type of fracture		column	shear

Schmidt hammer tests were also completed on the various concrete elements in the building. All tests were completed in zones remote from where the concrete cylinders were extracted from the building. The results from the Schmidt hammer tests are presented below.

The conversion from hardness information into concrete cylinder compressive strength is presented utilises the standard conversion factors typically use with Schmidt hammers, which has been derived from extensive testing on concrete samples in Europe. The results indicate that the normalised correlation curves typically overestimated the actual concrete strength when compared to the actual concrete strength information obtained from the concrete cores that were tested.

Table 7 Schmidt Hammer results for the Precast beams in Westpac Centre

location: Precast Beam

	1	**2**	**3**	**4**
A	65.5	53	56	56.5
B	57	63	56.5	54.5
C	54	62	58.5	58
D	67	60	45.5	52

Correct Average:	56.8	
Cube Strength:	42.4	MPa
Cylinder Strength, f'c:	**34.0**	**MPa**

Table 8 Schmidt Hammer results for the Columns in Westpac Centre

location: Column Level 3

	1	**2**	**3**	**4**
A	57	64.5	58.5	56
B	56.5	63.5	63	54.5
C	61	58	56.5	60
D	58.5	56	64	57.5

Correct Average:	58.6	
Cube Strength:	46.2	MPa
Cylinder Strength, f'c:	**37.0**	**MPa**

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Figure 5 *Core Drilling in concrete column*

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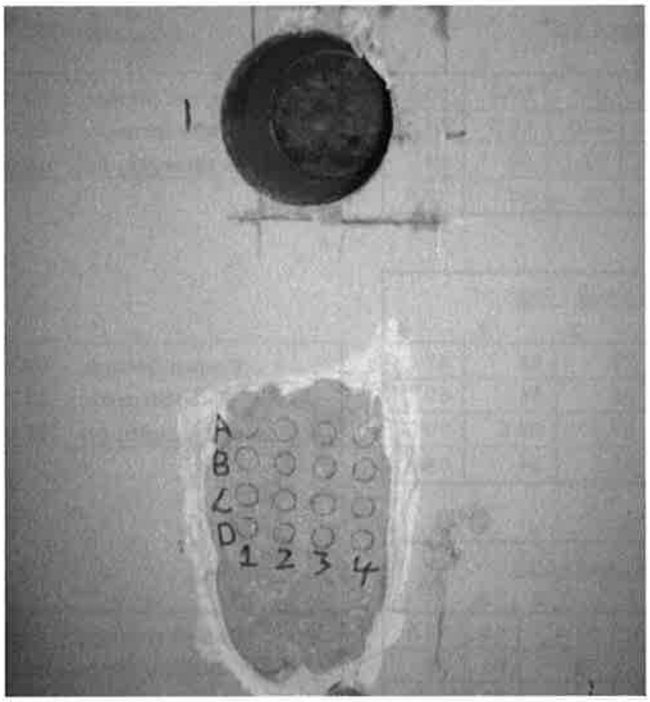

Figure 6 *Core and Schmidt hammer location on Wall element*

Figure 7 *Core location on Wall element*

Table 9 Schmidt Hammer results for the In-situ Wall elements of Westpac Centre

location: Basement Wall

	1	2	3	4
A	68.5	64	60.5	60.5
B	67	620	65.5	64
C	58	57	68	64
D	61.5	58	65	62.5

Correct Average:	62.9	
Cube Strength:	56.9	MPa
Cylinder Strength, f'c:	**46.0**	**MPa**

location: Level 3 wall - RHS

	1	2	3	4
A	67	71	68	57
B	56	56	54	62
C	66	57.5	59.5	70.5
D	61	63.5	54	69.5

Correct Average:	62.1	
Cube Strength:	55.7	MPa
Cylinder Strength, f'c:	**45.0**	**MPa**

location: Level 3 wall - LHS

	1	2	3	4
A	57.5	61.5	66	66.5
B	59	52	66	73
C	55.5	61	52	55.5
D	53	60.5	58	57

Correct Average:	59.8	
Cube Strength:	49.6	MPa
Cylinder Strength, f'c:	**40.0**	**MPa**

5.2. STEEL RESULTS

Four 16 mm diameter reinforcing bars were removed from the insitu concrete walls of the structure and subjected to uniaxial tensile testing in the laboratory. Two of the bars were retrieved from areas in the building considered to have sustained little or no damage during the recent earthquakes. As such the material properties obtained from these sample can be assumed to have been unmodified from previous inelastic strain cycles. One of the bars was from the horizontal reinforcing and the other formed an element of vertical reinforcing in the wall

The obtained stress-strain responses of the two undamaged steel samples are shown in *Figure 10* below. The steel samples were subjected to unidirectional cyclic tensile testing rather than cycles of reverse cyclic loading to near equal values of tensile and compressive strain. In the structural element, under imposed lateral loads the neutral axis is likely to have been located near the location of the reinforcing steel during the compression load cycle, and as such the steel would have been subjected to very small induced compressive strains. During the reverse loading cycle the steel located at or near a crack in the concrete section is likely to have been subjected to disproportionately larger tensile strains, thereby significantly skewing the strain profile experienced by the reinforcing steel into the tension domain. Due to the skewed strain profile, it is believed that the unidirectional cyclic tensile test provides an adequate representation of the strains induced in the steel during a seismic event.

Leeb Hardness testing was also completed on the steel samples at various levels of applied strain, both with the load applied and with the load removed from the steel. The points of inspection can be observed in the recorded stress-strain response as areas of load cycling.

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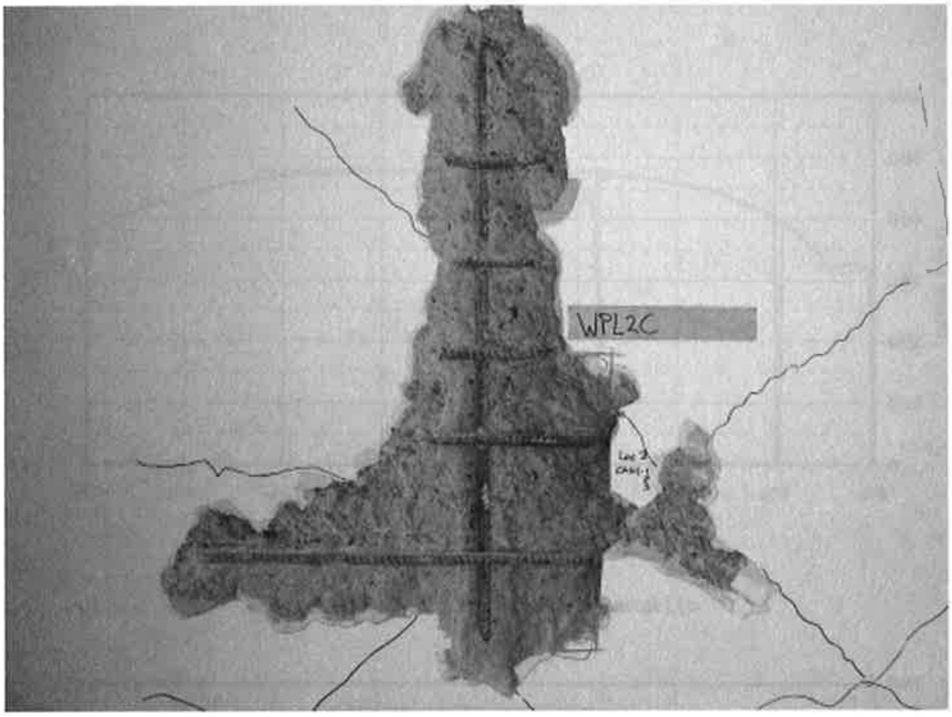

Figure 8 *Exposed reinforcing steel in zone of damage in wall element*

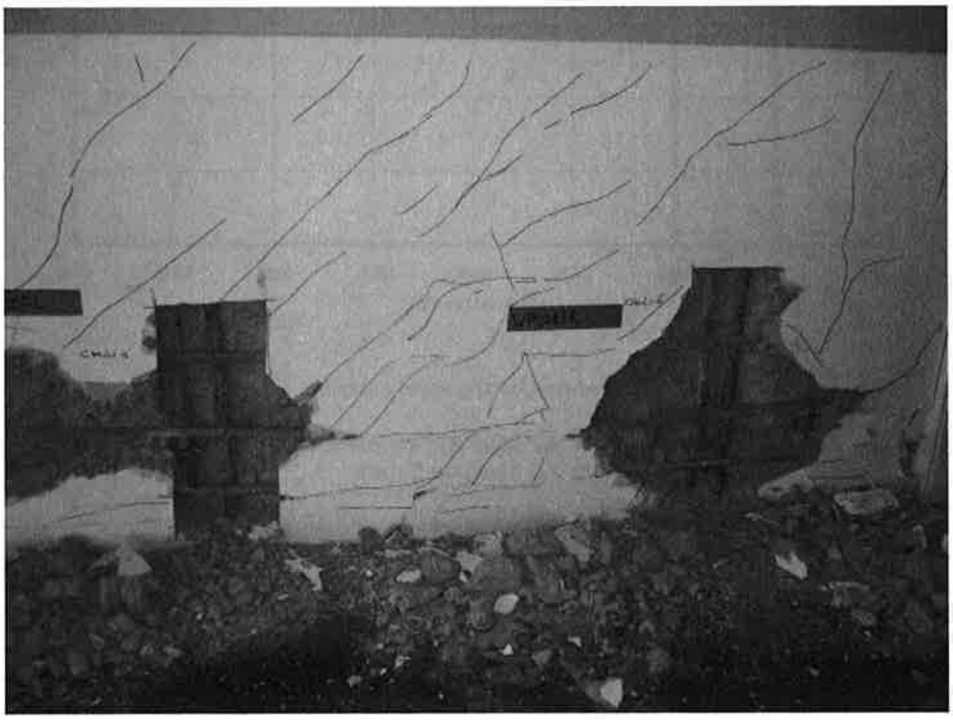

Figure 9 *Exposed reinforcing steel in zone of damage in wall element*

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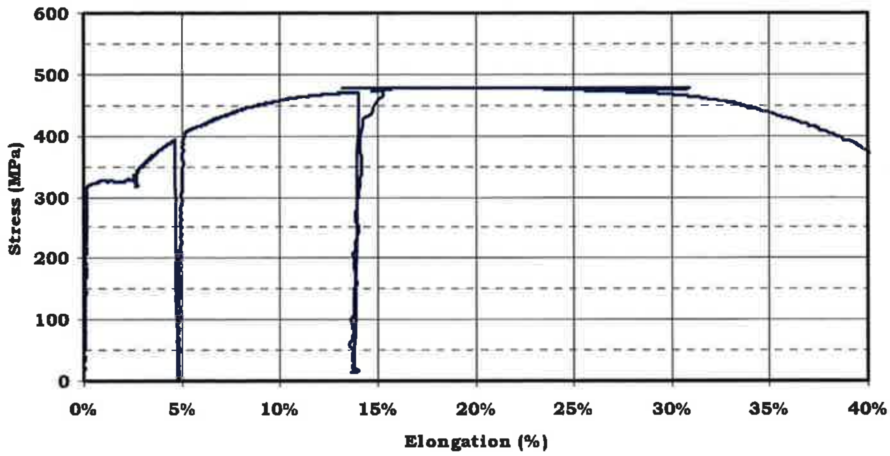
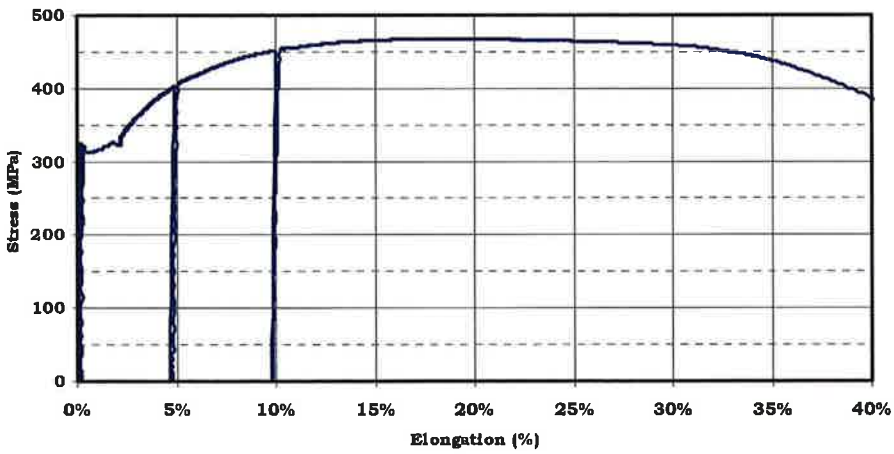


a) Horizontal reinforcing bar sample



b) Vertical reinforcing steel sample

Figure 10 Materials Test Result for the Steel test coupons obtained from undamaged area in the Westpac Centre

The steel samples had an average recorded yield stress (f_y) of 320 MPa and an average maximum recorded stress (f_u) of 472 MPa. The strain hardening ratio (f_u/f_y) of the tested steel sample was defined as 1.475. This value of strain hardening ratio indicates that the steel has a good likelihood of spreading the zone of yield along the bar, a beneficial property for limiting the potential damage at a localised zone of damage in a reinforced concrete member. It also indicates that the steel has a high plastic hardness and therefore it likely to provide suitable variation in Leeb hardness values for various levels of imposed strain.

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The recorded Leeb hardness for the steel samples, and the associated stress and strain at the point of testing are reported below. A series of 6 individual Leeb hardness test results were taken and averaged to produce the reported value of Recorded Average Leeb. The recorded Leeb values for the steel show a good variation across the stress range. This is a result of the relatively high plastic stiffness of the material, defined by the extent of strain hardening observed in the recorded stress-strain plot of the tested samples.

The reported values of Leeb hardness were derived for the steel sample supported in the universal testing machine. Additional hardness tests were also completed on the tested steel sample with the bar fully supported in a mortar matrix. Based on the Leeb Hardness results obtain, the reinforcing steel used in the building appears to have a base Leeb Hardness of 610 DLHL.

Table 10 Baseline Material Strength Results for Test Sample 1

Applied Load (kN)	Steel Strain (%)	Steel Stress (MPa)	Recorded Average Leeb (DLHL)
0.0	0.0	0	610
61.0	0.5	303	610
80.0	5.0	398	650
95.0	14.0	472	680

Table 11 Baseline Material Strength Results for Test Sample 2

Applied Load (kN)	Steel Strain (%)	Steel Stress (MPa)	Recorded Average Leeb (DLHL)
0.0	0.0	0	610
63.0	0.5	313	612
80.0	4.5	398	650
91.0	11.0	453	670

Leeb Hardness testing was completed on a further 2 horizontal bar and two vertical bar located in zones of heavy damage in the in-situ wall of the building. The results from the Leeb Hardness are presented below.

The Leeb hardness results for the Vertical Bar 2 shows a peak elevated hardness value of 660 DLHL approximately mid way along the length of tested steel. This zone of elevated hardness coincides with the location where the reinforcing bar crosses a significant crack in the wall element. The zone of elevated hardness occurs over a length of approximately 35-40 mm, equivalent to 2 times the diameter of the reinforcing bar. Based on the derived correlations obtained from the undamaged reinforcing bars, this level of Leeb Hardness indicates that the steel has previously been strained to approximately 10% strain. This level of induced strain indicates that the steel has lost approximately 75% of the available strain capacity, and can only undergo an additional 5% strain before fracturing. Based on the short zone observed to have an elevated hardness, this would equate to approximately 2 mm of elongation over a 40 mm length prior to fracture.

The Leeb hardness for the Horizontal Bar 2 shows signs of moderately increased strain hardening over lengths of approximately 75-100 mm. Based on the correlations between Leeb Hardness and strain obtained previously, it is suggested that this steel sample has been previously strained to 2%.

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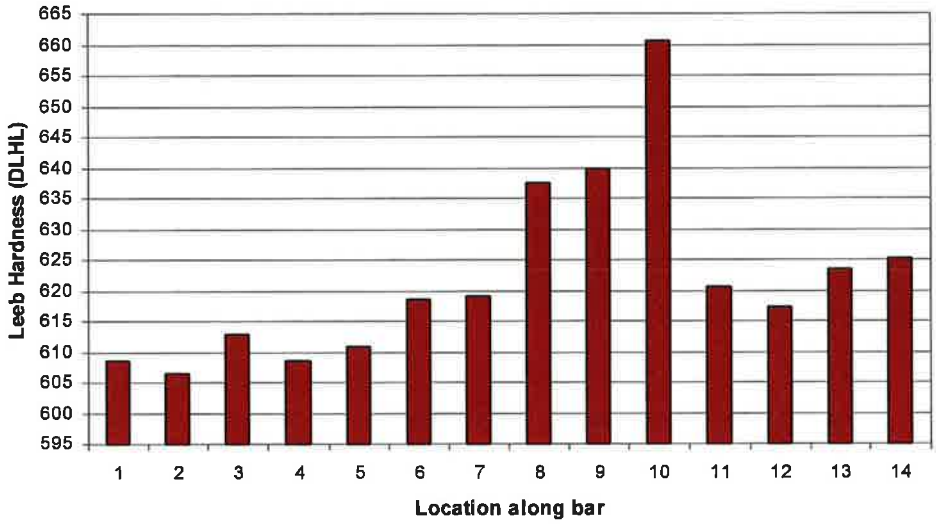

Figure 11 Leeb Hardness result for Vertical Bar 2 in zone of damage

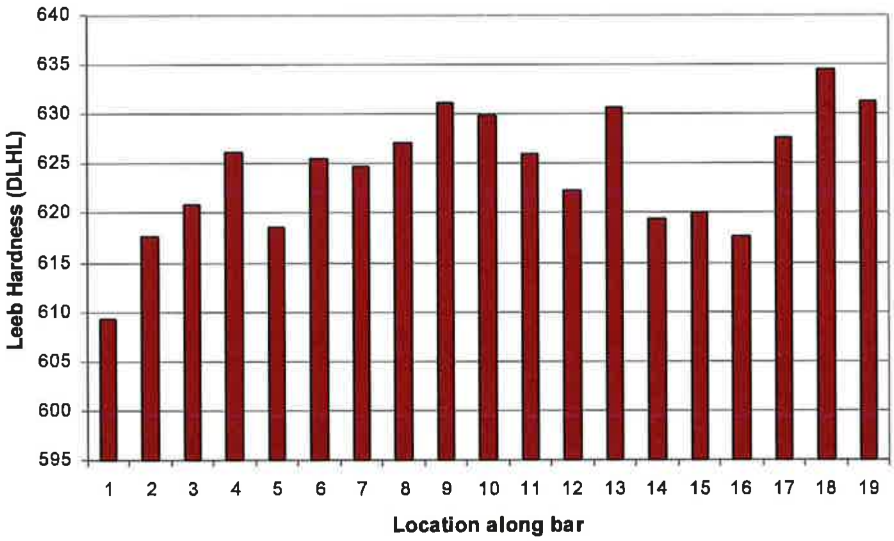

Figure 12 Leeb Hardness result for Horizontal Bar 2 in zone of damage

Vertical reinforcing Bar 3 shows two zones of increased hardness, corresponding to two cracks observed to cross the steel in the wall element. The first zone of elevated hardness is relatively wide, indicating that any yielding of the steel occurred across a relatively long length on the bar. The second zone of elevated hardness has a maximum recorded Leeb value of 640 DLHL and appears to occur over a relatively short distance. This level of hardness indicates that the steel was previously strained to approximately 5%. The results for Horizontal Bar 3 are similar to the previous horizontal bar with Leeb hardness values suggesting the steel was subjected to inelastic strains of approximately 2% over a relatively long length of the steel.

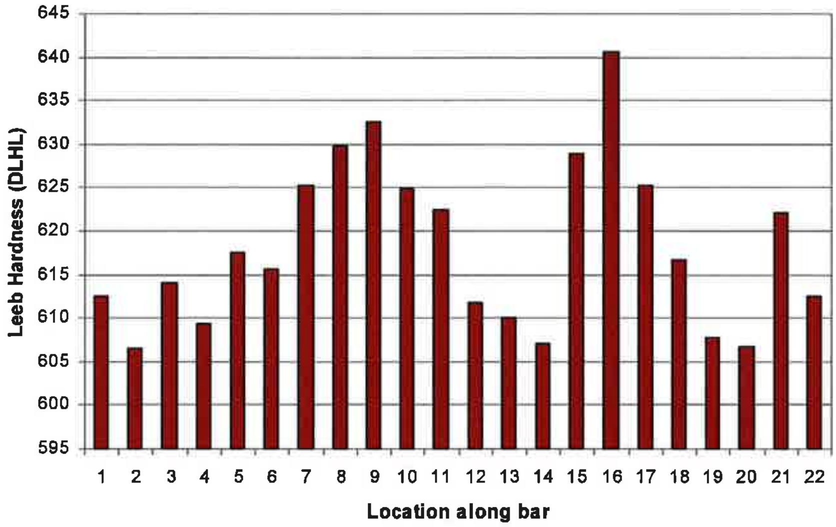

Figure 13 Leeb Hardness result for Vertical Bar 3 in zone of damage

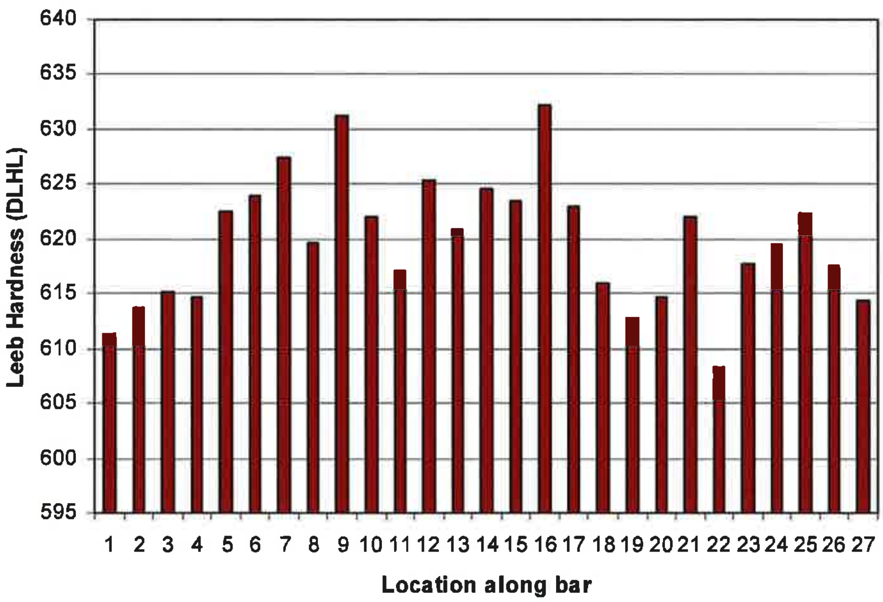

Figure 14 Leeb Hardness result for Horizontal Bar 3 in zone of damage

A further 4 reinforcing bar samples were removed from the building and subjected to destructive tensile testing. The results from the testing are shown below. The results indicate that the horizontal steel remained undamaged during the earthquake, with recorded uniform strain capacities in excess of 33%. The yield strength of the tested horizontal steel samples was found to be 314 MPa and 315 MPa respectively.

The vertical steel sections were found to have considerable lower uniform elongation capacity when compared to the horizontal steel section, with actual elongation capacities between 11% and 13%. This result indicates that the steel has lost strain capacity due to being exposed to previous cycles of inelastic loading. The yield strength of the vertical steel sections was found to be 319 MPa and 330 MPa respectively.

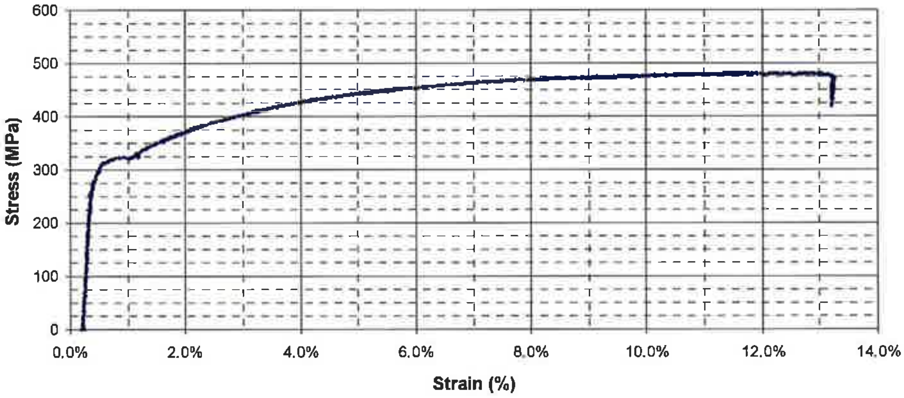

Figure 15 Stress-strain response for vertical steel section located in damaged zone

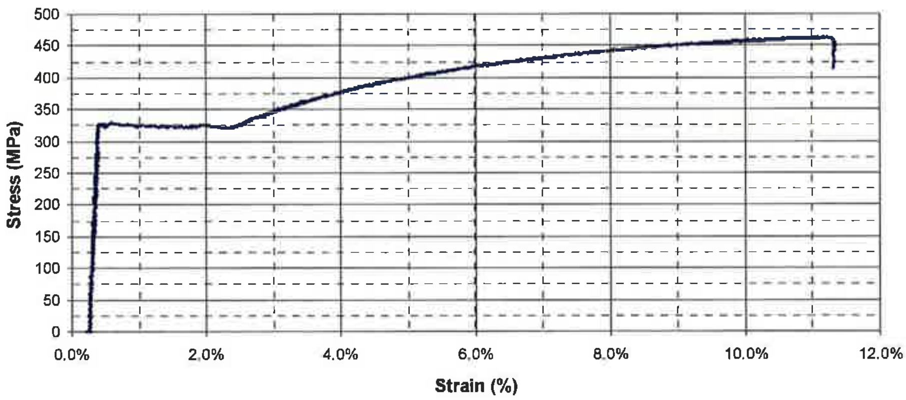

Figure 16 Stress-strain response for vertical steel section located in damaged zone

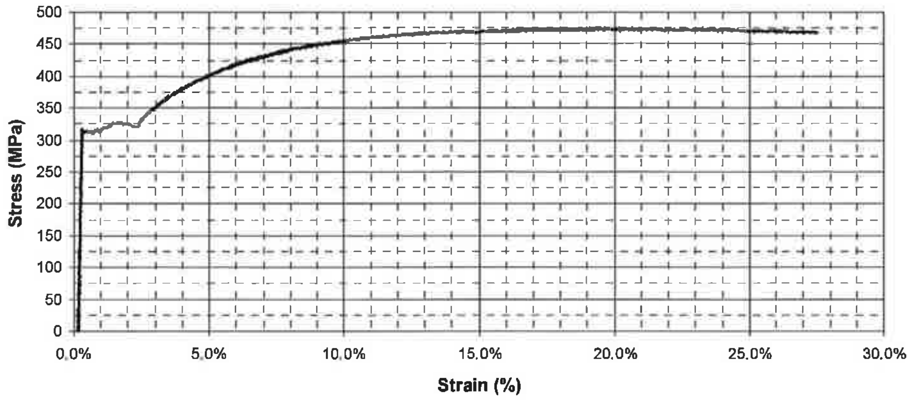

Figure 17 Stress-strain response for horizontal steel section located in damaged zone

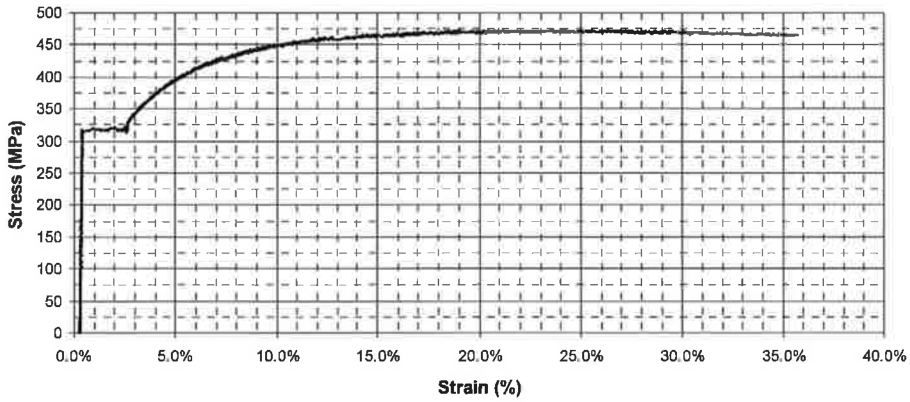

Figure 18 Stress-strain response for horizontal steel section located in damaged zone

6.0 IRD BUILDING RESULTS

6.1. CONCRETE RESULTS

No concrete cores were extracted from the IRD building. As a result, all concrete material information was obtained from Schmidt hammer tests. All tests were completed near the zones of damage in the in-situ and precast concrete shear walls. The results from the Schmidt hammer tests are presented below.

The conversion from hardness information into concrete cylinder compressive strength is presented utilises the standard conversion factors typically use with Schmidt hammers, which has been derived from extensive testing on concrete samples in Europe.

Table 12 Schmidt Hammer results for the Precast Walls in the IRD Building

location: Precast Wall section -1

	1	2	3	4
A	69.5	63.5	61.5	60.5
B	66	56.6	65.6	62.1
C	57	68.5	68	62
D	67	68	63	61.5

Correct Average:	64.1	
Cube Strength:	60.1	MPa
Cylinder Strength, f'c:	**48.0**	**MPa**

location: Precast Wall section -2

	1	2	3	4
A	72	63.5	68	69
B	68.5	63	58.5	61,5
C	56.5	63.5	59.5	71
D	63	65.5	58.5	72

Correct Average:	64.3	
Cube Strength:	61.2	MPa
Cylinder Strength, f'c:	**49.0**	**MPa**

location: Insitu Wall section -1

	1	2	3	4
A	58	59.5	63	63.5
B	65	63.5	67	71
C	53.5	55	52.5	53.5
D	55.5	65	57	60

Correct Average:	59.8	
Cube Strength:	50.1	MPa
Cylinder Strength, f'c:	**40.0**	**MPa**

location: Insitu Wall section -2

	1	2	3	4
A	65	61	59.5	55.5
B	55.5	65	56	63.5
C	65.5	66	55.5	59.5
D	58.5	59	62	55

Correct Average:	60.3	
Cube Strength:	50.2	MPa
Cylinder Strength, f'c:	**40.0**	**MPa**

6.2. STEEL RESULTS

HSL was commissioned independently to undertake materials testing on two steel samples extracted from the concrete walls of the IRD building. Two deformed reinforcing bars, 10 mm in diameter, were supplied for testing. The location of the steel in the building nor the origins of the steel were provided.

Prior to undertaking uniaxial tension testing on the steel, the samples were subjected to Leeb Hardness testing. The obtained results are presented below.

Both steel samples showed significant reduction in Leeb hardness readings at the location marked on the bars as corresponding with the crack in the concrete member. Reduction in Leeb hardness typically only occurs in steel bars immediately prior to the onset of necking, where micro alloy steel has been found to strain soften.

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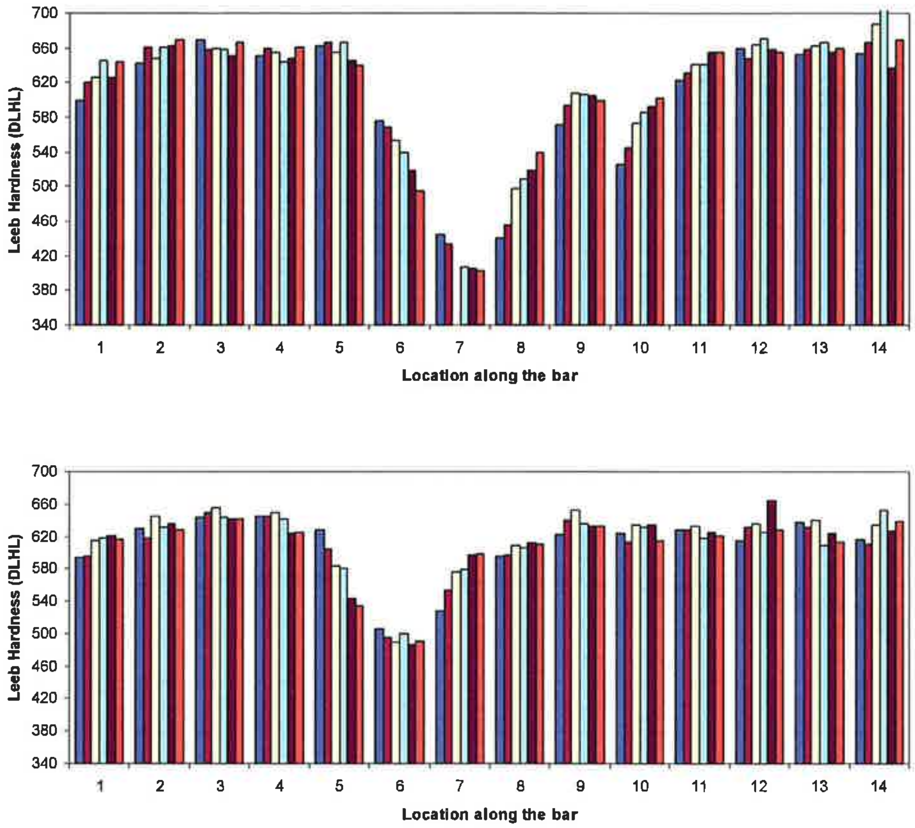

Figure 19 Leeb Hardness result for steel samples provided from IRD Building

The steel samples were then subjected to uniaxial tensile testing, with the obtained stress-strain responses shown in *Figure 20*. From the obtained stress-strain responses it would appear that the parent material was Grade 300E reinforcing steel. Grade 300E reinforcing steel has a lower characteristic yield strength of 300 MPa and is required in the New Zealand manufacturing Standard (AS/NZS 4671) to have a minimum uniform elongation capacity in excess of 15%. The results obtained for the two samples show they have an elongation capacity of 2% and 0.9% indicating that they have undergone significant inelastic deformation and are close to fracturing. This correlates with the observed Leeb Hardness results, showing significant strain softening at the cracked region.

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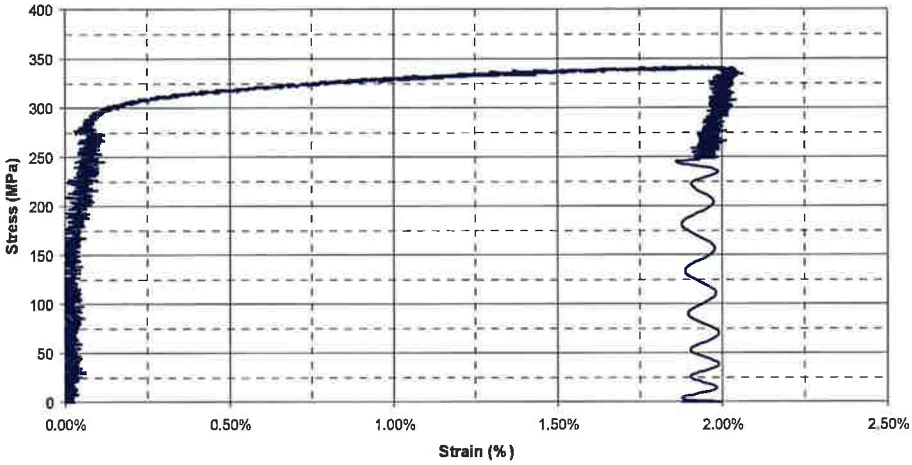

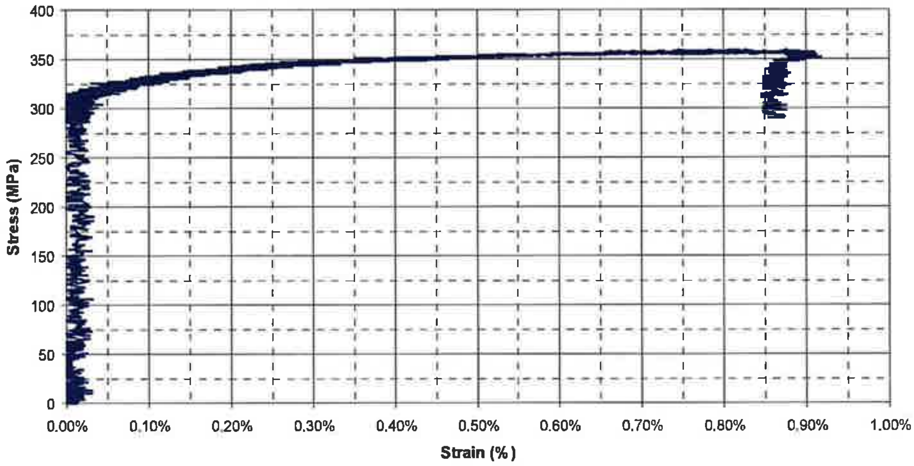

Figure 20   Stress-Strain responses for steel samples provided from IRD Building

Figure 21 Tensile testing of steel sample

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7.0 BIBLIOGRAPHY

[A1] ASTM International, "*Standard Test Method for Indentation Hardness o Metallic Materials by Portable Hardness Testers*", ASTM Designation E110-10, 2010

[A2] ASTM International, "*Standard Test Method for Leeb Hardness Testing of Steel Products*", ASTM Designation A956-06, 2006

[B1] Bonora, Gentile, Pirondi, Newaz, "*Ductile damage evaluation under triaxial state of stress: theory and experiments*", International Journal of Plasticity 21, 2005, pg 981-1007

[B2] Bonora, Newaz, "*Low cycle fatigue life estimation for ductile metals using a nonlinear continuum damage mechanics model*", International Journal of Solids and Structures, Vol 35, No 16, pp 1881-1894, 1998

[G1] Gasko, Rosenburg, "*Correlation between Hardness and Tensile Properties of Ultra High Strength Dual Phase Steels*", Materials Engineering, 18, pg 155-159, 2011

[K1] Kapoor, "*A re-evaluation of the life to rupture of ductile metals by cyclic plastic strain*", Fatigue and Fractures of Engineering Materials and Structures, Vol 17, No 2, pp 201-219, 1994

[L1] Lemaitre Dufailly, "*Damage Measurements*", Engineering Fracture Mechanics, Vol 28, pp 643-661, 1987

[M1] Mkaddem, Gassara, Hambli, "*A new procedure using micro-hardness technique for sheet material damage characterisation*", Journal of Material Processing Technology, 178, 2006, pg 111-118

[M2] Matsumoto, Y, "*Study on the residual deformation capacity of plastically strained steel*", Yokohama National Library, 2009

[N1] Nishikawa, Soyama, "*Two step method to evaluate equibiaxial residual stress of metal surface based on micro-indentation tests*", Materials and Design 32, 2011, 3240-3247

[N2] Nakane, Kanno, Kurosaki, Takagi, Komotori, Ogawa, "*Accuracy of Plastic Strain Estimated by Hardness to assess remaining Fatigue Live*", Material Propertied Measurement, 2010.

[N3] Nomoto, " *Report on the Integrity Assessment f Structures, Systems, and Components of the KK-NPP by the JANTI Committee*", 20th International Conference on Structutal Mechnaics in Reactor, Technology, 2009

[P1] Pirondi, Bonora, "*Modelling ductile damage under fully reversed cycling*", Computational Materials Science 26, 2003, pg 129-141

[T1] Tasan, Hoefnagels, "*A critical assessment of indentation based ductile damage quantification*", Acta Materialia, Volume 57, Issue 17, October 2009, Pages 4957, 4966

[T2] Tasan, Hoefnagels, Geers, "*Indentation-based damage quantification revisited*", Scipta Materialia 63, 2010, pg 316-319

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