PYNE GOULD

PYNE GOULD CORPORATION BUILDING
SITE EXAMINATION AND MATERIALS TESTS

FOR DEPARTMENT OF BUILDING AND HOUSING

17TH SEPTEMBER 2011

Hyland

FATIGUE + EARTHQUAKE ENGINEERING

REPORT PREPARED FOR:

DEPARTMENT OF BUILDING AND HOUSING

REPORT PREPARED FOR:

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EXECUTIVE SUMMARY

The Pyne Gould Corporation Building collapsed during the After-shock on 22nd February, 2011. The building was largely deconstructed, leaving a pile of debris on the site, by the time of the site examination undertaken by Hyland Consultants Limited ("HCL") for the purposes of this report ("the Site Examination").

Structural remnants were recovered from the debris for examination on 16th March 2011. Their configuration and condition were documented, and samples were taken for testing to allow further engineering studies to be conducted to better understand why the building collapsed.

The remnants examined included two steel jacketed concrete columns, 250 x 250 mm square concrete columns, and various beam and slab items.

One steel jacketed column had a steel cap plate with fractured fillet welds that appear to have connected a 305 mm wide concrete encased steel girder it had supported. This form of construction could be seen at Level 2 on the remaining portion of structure that was still standing. The concrete at the other end of the column had been crushed and the reinforcing steel bars necked and fractured.

The other steel jacketed column contained necked and fractured reinforcing at each end where the column ends had been embedded into concrete members above and below. One longitudinal bar had torn through the steel jacketing and the internal tie bars.

A deep concrete beam remnant was found that had a crushed beam to column concrete zone, with necked and fractured reinforcing in a pattern matching that seen on the circular steel jacketed concrete columns.

The reinforcing steel typically had nominal diameters consistent with it being made to imperial measurements. However some Japanese sourced 24 mm bar appeared to be made to metric dimensions.

Reinforcing steel tensile tests and chemical analyses showed that the #4, #5, #8 and the Japanese D24 reinforcing steel had properties consistent with NZS 1693:1962 Grade 33 ksi (228 MPa). They also had tensile properties close to the minimum requirements of modern day AS/NZS 4671:2001 Grade 300E.

However chemical analysis of the #8 bar sample showed it to have reasonably high levels of tin (Sn) and phosphorus (P) that would have raised its transition temperature and made it more susceptible to notch initiated fracture.

The larger diameter #10 (31.8 mm) bars appear to be Grade HY60 ksi (413 MPa) bars conforming to NZS 1879:1964. These bars had high carbon and alloy contents and associated carbon equivalents which made them very susceptible to cracking when welded or notched. During tensile testing some of these bars developed running flat fracture after initially yielding but prior to attaining their ultimate tensile strength. Two examples of this form of fracture were observed on site but these fractures may have occurred during de-construction based on the nature of the damage to the bars in the vicinity of the fractures.

Concrete cores were extracted and tested from two members. The average compressive strength of those extracted from a 250 x 250 mm column was 47.3 MPa. This was greater than the specified 28 day strengths for columns of both 3500 psi (24.1 MPa) and 4000 psi (27.5 MPa) specified dependant on column location in the building. The expected test strengths accounting for grade variation and strength-aging were 36 MPa and 41 MPa respectively.

The average compressive strength of cores extracted from a 700 x 510 mm beam was 40.7 MPa. This is greater than the specified 28 day strength for beams of 2500 psi (17.2 MPa). The expected test strength accounting for grade variation and strength-aging was 26 MPa.

The beam and column concrete tested therefore met the specified strengths at the time of the collapse. The concrete tested also met the expected strengths with allowance for grade variation and strength-aging, indicating that the concrete met the specified strength requirements at the time of construction.

Disclaimer:

The observations made in this report cover only a sample of structural remnants able to be accessed at the time of the Site Examination. They therefore need to be interpreted in conjunction with the original structural design drawings and specification, and modifications that may have occurred prior to the After-shock, as well as photos and observations of the structure immediately after the After-shock and during its subsequent de-construction. Some of the damage observed and documented in this report may have occurred during deconstruction.

1. INTRODUCTION

A. OBJECTIVE

The objective of this report was to document the configuration and the condition of structural remnants from the debris that may assist in the analysis of the building's collapse during the earthquake after-shock on 22nd February, 2011 ("the After-shock").

B. SCOPE

The Department of Building and Housing set out the following scope upon which this report has been prepared:

- Seek out relevant drawings of the structure from the Christchurch City Council.
- Access the site and pull out structural remnants from the debris for examination using a mobile crane.
- Layout and visually examine and document structural remnants.
- Remove samples of reinforcing steel and concrete cores for code conformance checks and possible back engineering of the collapse condition.
- Report on findings.

C. BACKGROUND

The Pyne Gould Corporation Building was a multi-level reinforced concrete building with cast in-situ beams, columns, floors and shear walls, with in some locations concrete encased steel beams supported on concrete filled steel jacketed tube columns.

It was designed in 1963. No drawings of the building were available at the time of the Site Examination.

The building was severely damaged in the After-shock and partially collapsed. It was then deconstructed leaving the site covered in building debris at the time of the Site Examination.

2. EXAMINATION OF REMNANTS OF STRUCTURAL COMPONENTS

The examination of structural remnants was undertaken on Wednesday 16th March, 2011.

The site was identified as containing asbestos which required the wearing of asbestos personal protection (Figure 1).

A crane was used to move and extract selected structural remnants form the debris.

The structure was built in the pre-metric time of the 1960s, so in this report the bar designations have been recorded as imperial bar types using the # system with bar numbers a multiple of 1/8 inch ie #4 = ½" diameter. Where measurements were taken on site with metric vernier callipers these are noted in brackets.

The following observations were made during this site inspection.

Figure 1 – Structural concrete beam remnant being moved by mobile crane and site crew wearing personal protection equipment due to presence of asbestos.

A. ITEM 1: 2350 X 250 MM 4-BAR CONCRETE COLUMN

This 250 x 250 mm concrete column had 12 mm solid plaster render on each face. It was possibly continuous over two levels with a 2200 mm portion designated W, separated by an 800mm gap to a 2970 mm portion (Figure 2).

The W portion had four #8 bars, two of which had bamboo deformations and a "JAPAN" rolling mark. The other two bars had diagonal deformations.

A bar with bamboo deformations measured 23.0 mm diameter using callipers. One with diagonal deformations measured 24.7 mm diameter. The bamboo bars therefore appear to have been metric 24 mm bars sourced from Japan, whereas the bars with diagonal deformations were #8 or 1" diameter bars. Samples of the Japanese bars were extracted for tensile testing and chemical analysis.

Three concrete cores were extracted for concrete testing.

The E portion had four #6 bars with #2 ties at 225 centres. The ties had 135° re-entrant bends.

E

W

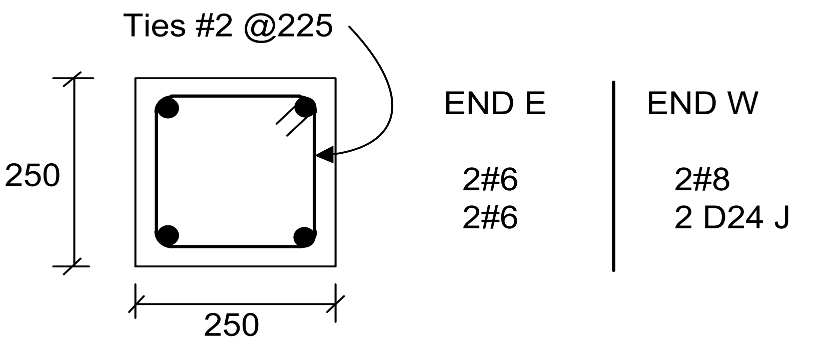

Figure 2 - Item 1 - 250 x 250 Concrete Column

B. ITEM 2: 410 MM DIAMETER STEEL JACKETED CONCRETE COLUMN

The steel jacketed concrete column was 2700 mm long and 410 mm diameter, with a 3.5 mm thick jacket.

The end 100 mm of the jacketing appeared to have been cast into concrete members and was similar to what was seen on the underside of the concrete beam Item 8A (Figure 10).

The column had seven #7 bars (22.8 mm) spaced around a circumference with #2 ties (6.6 mm) at 300 mm centres (Figure 3).

The longitudinal bars had two forms of deformation indicating different manufacturers.

Four bars had bamboo style deformations and had the rolling mark "JAPAN". These measured 23.0 diameter using vernier callipers. The other three had diagonal deformations and measured 24.7 mm in diameter. This indicates that the bamboo deformed bars were metric size D24 bars sourced from Japan, and the others were imperial size #8 bars.

One longitudinal bar had torn through the steel jacketing and the internal ties.

Five of the bars had necked and fractured and two had de-bonded at the end adjacent to the painted "2" marking on the beam.

At the other end all seven bars had necked and fractured.

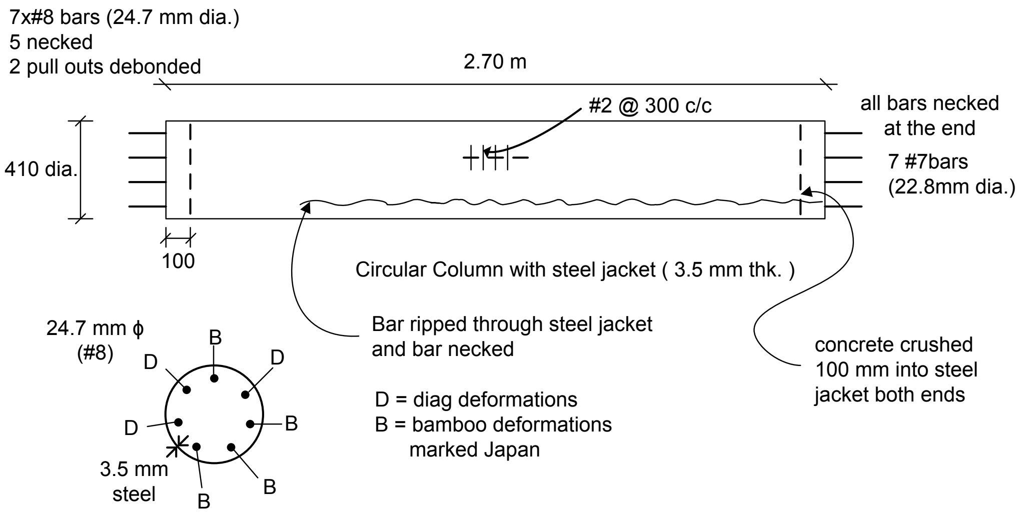

Figure 3 - Item 2 - 410 dia. Steel Jacketed Concrete Column; (photos clockwise from top left) (a) Torn steel jacketing; (b) Numbered end of column with 5 fractured and 2 de-bonded bars; (c) Other end with 7 necked and fractured bars and crushed concrete zone

C. ITEM 3: 410 MM DIAMETER STEEL JACKETED AND CAPPED CONCRETE COLUMN

The steel jacketed concrete column was 3940 mm long and 410 mm diameter, with a 3.5 mm thick jacket. The end 100 mm of the jacketing appeared to have been cast about 100 mm into a concrete supporting member. The column had twelve #7 bars (22.6 mm) (Figure 4).

At the end of the column, adjacent to the painted "3" marking, which appeared to be the column base, all of the bars had necked and fractured.

The steel jacket had burst on one side and the concrete in that area crushed further up the column than the other side, indicating that compressive flexural failure occurred, rupturing the steel jacket.

A 380 x 380 x 19 mm cap plate was fixed to the top of the column. Two lines of 8 mm fillet weld spaced 305 mm apart had fractured where they connected what is thought to have been a steel welded girder, similar to those still in place at the time onto the remaining portion of Level 2 structure that was still standing (Figure 13).

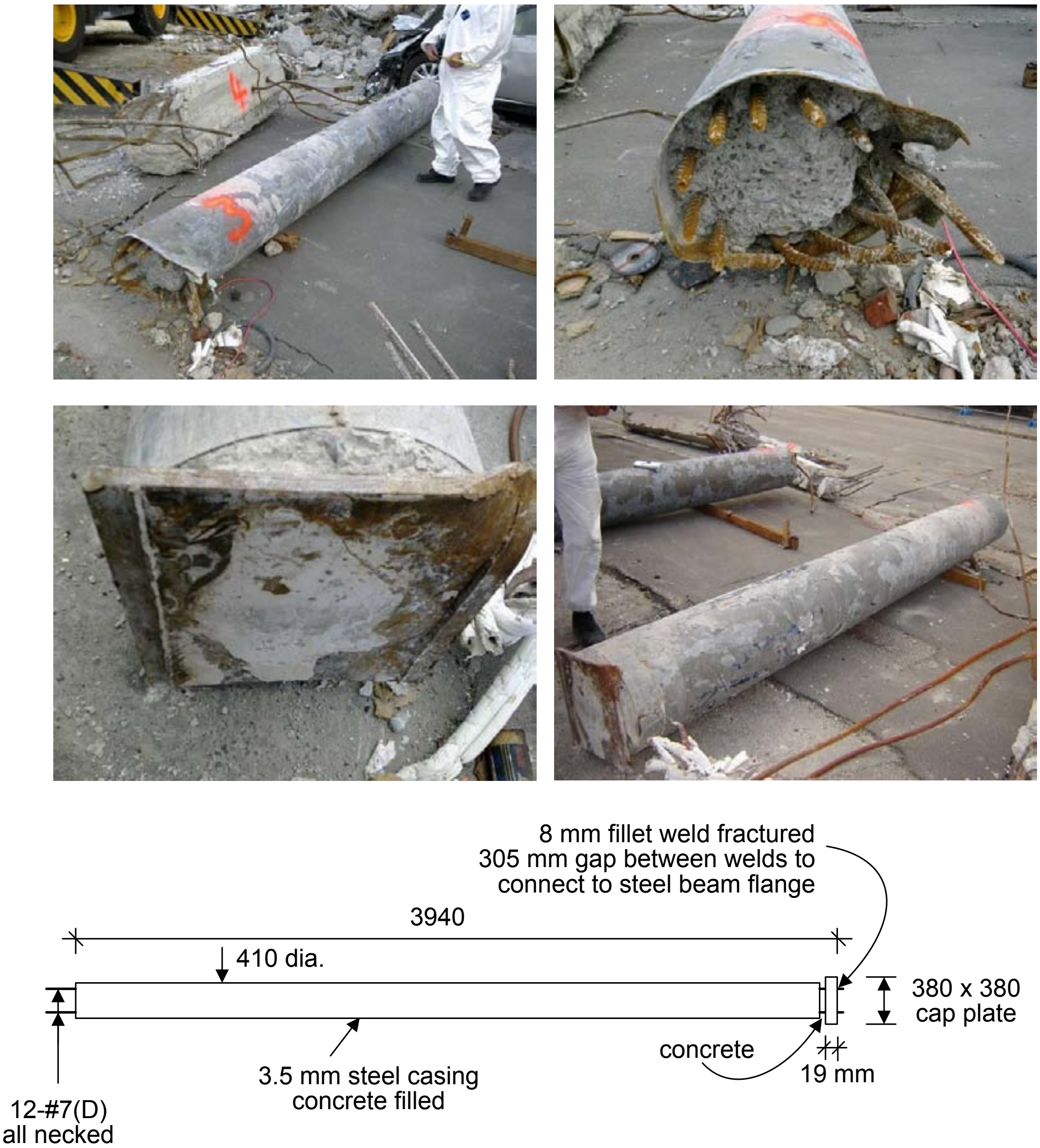

Figure 4 - Item 3 -410 dia. Steel Jacketed Concrete Column with Cap Plate (photos clockwise from top left): (a) Column bottom end; (b) Burst jacketing on one side with deep crushed concrete zone and necked and fractured longitudinal bars; (c) Top end with cap plate; (d) Cap plate with two lines of fractured fillet weld 305 mm apart.

D. ITEM 4: 700 X 510 MM REINFORCED CONCRETE BEAM

This 700 x 510 mm concrete beam remnant had five #10 (30.5 mm) bottom bars, two of which had flat fractures (Figure 6). The two fractured ends were removed for examination. This showed that the fractures most likely occurred during de-construction, initiating at heavy abrasion marks and localised deformations on the bar surfaces from that process (Figure 6).

These bars were found during laboratory testing by SAI Global (NZ) Ltd to be of higher strength than the other bars and conformed to NZS 1879:1964 Grade HY60 (414 MPa) (SNZ 1964).

During tensile testing some fractured suddenly with a flat fracture surface similar to that seen on site after yielding, but prior to attaining their potential ultimate tensile strength.

On the top of the beam there were two #8 (25.4mm) bars that had de-bonded and two #4 (14.3 mm) that had necked and fractured.

Stirrups were #4 (12.9 mm) bars at 125 mm centres.

There was hairline diagonal cracking in the beam sides.

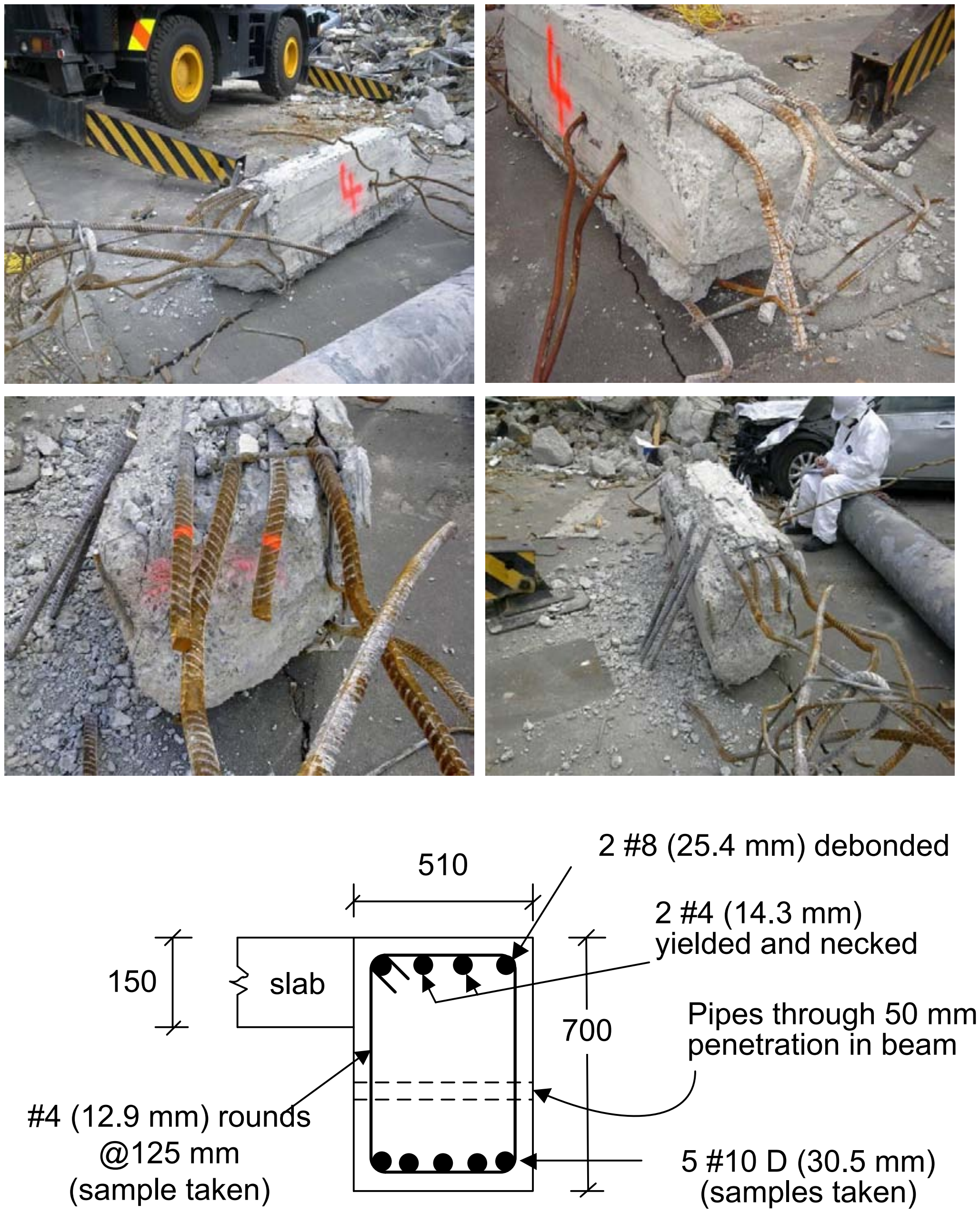

Figure 5 - Item 4 - 700 x 510 Concrete Beam (photos clockwise from top left): (a) Front with slab side down; (b) Rear; (c) Bar extracted; (d) Marked #10 bars with flat fracture surfaces

Figure 6 - Item 4 - Orange marked #10 bars samples shown in Figure 5 with flat running fractures initiating from surface abrasion marks likely caused during deconstruction.

E. ITEM 5: 700 X 510 MM BEAM WITH 500 X 350 TRANSVERSE BEAM

This 700 x 510 mm concrete beam was 5100 mm long with a 500 x 350 mm deep transverse beam 2000 mm long attached at one end (Figure 7).

Black bituminous waterproofing remains were found on the top surface.

The #4 slab topping bars at 250 mm centres had de-bonded from the top of the beam.

A 150 mm slab remnant that was attached is described in Item 6.

Large diagonal shear cracks had opened up on the side of the beam and the 135^{o} re-entrant stirrup hooks had partially straightened to around 100^{o} in the proximity of the crack.

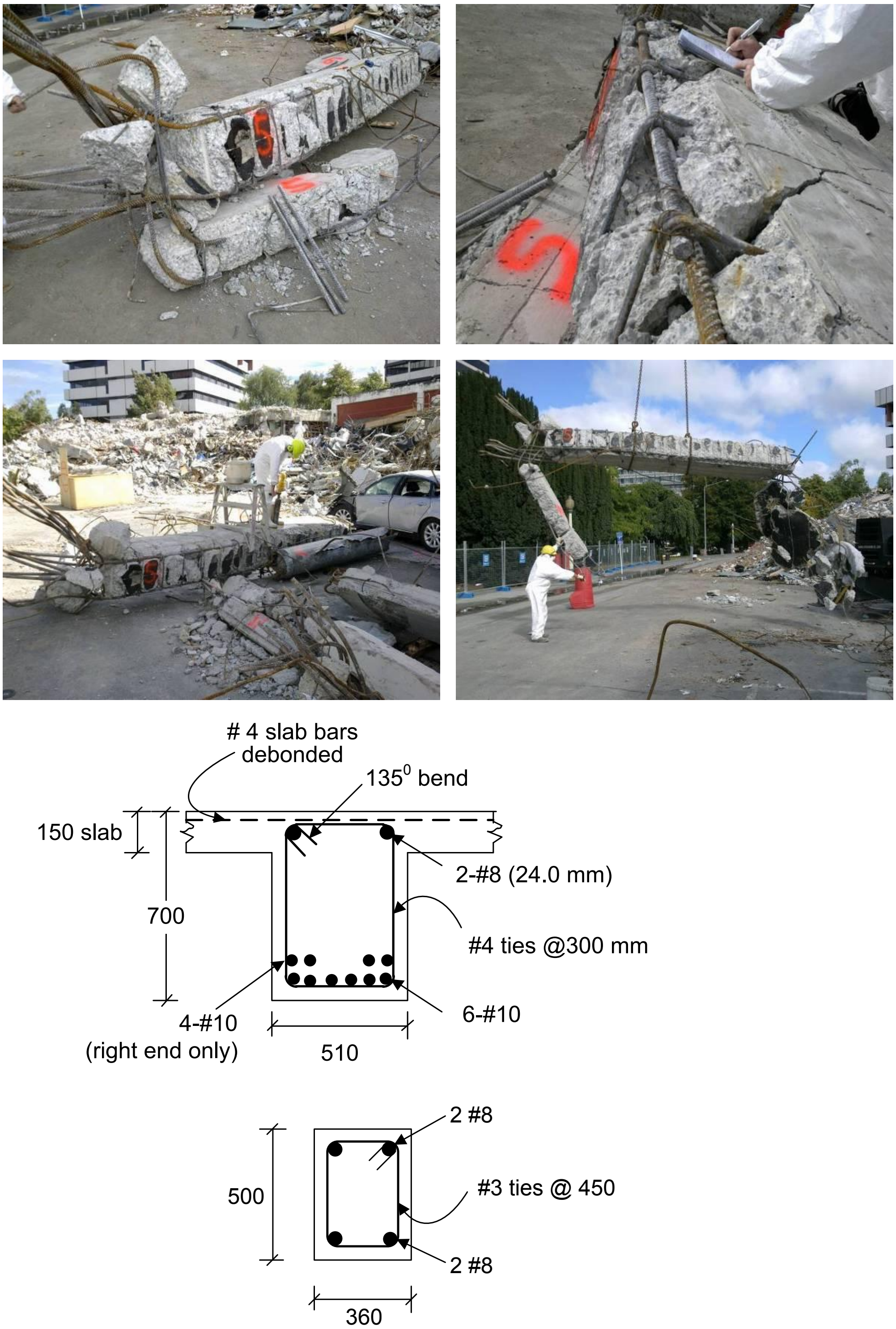

Figure 7 - Item 5 - 700 x 510 mm Concrete Beam and 500 x 350 mm Transverse Beam (photos clockwise from top left): (a) Top of concrete surface showing water proofing membrane remains; (b) Deep diagonal shear cracks in beam side with stirrup 135° re-entrant bend having partially straightened to around 100°; (c) beam being moved with transverse beam hanging down; (d) Concrete cores being drilled for compression testing.

F. ITEM 6: 150 MM REINFORCED CONCRETE SLAB

This portion of in-situ concrete slab was attached to the 700 x 510 mm concrete beam described as Item 5 (Figure 7).

The reinforcing consisted of #4 or #5 bars at 250 or 350 mm centres each way on the faces. These had "Pacific Steel" roller markings.

Some bars were extracted for tensile testing and chemical analysis.

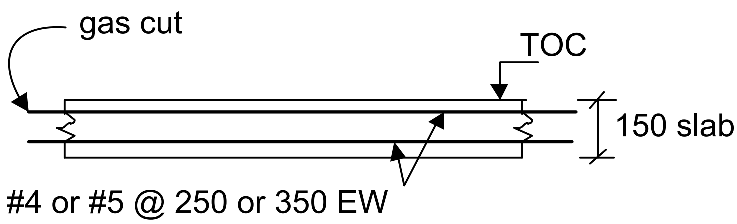

Figure 8 - Item 6 - 150 mm Slab

G. ITEM 7: 610 X 275 MM REINFORCED CONCRETE SPANDREL BEAM

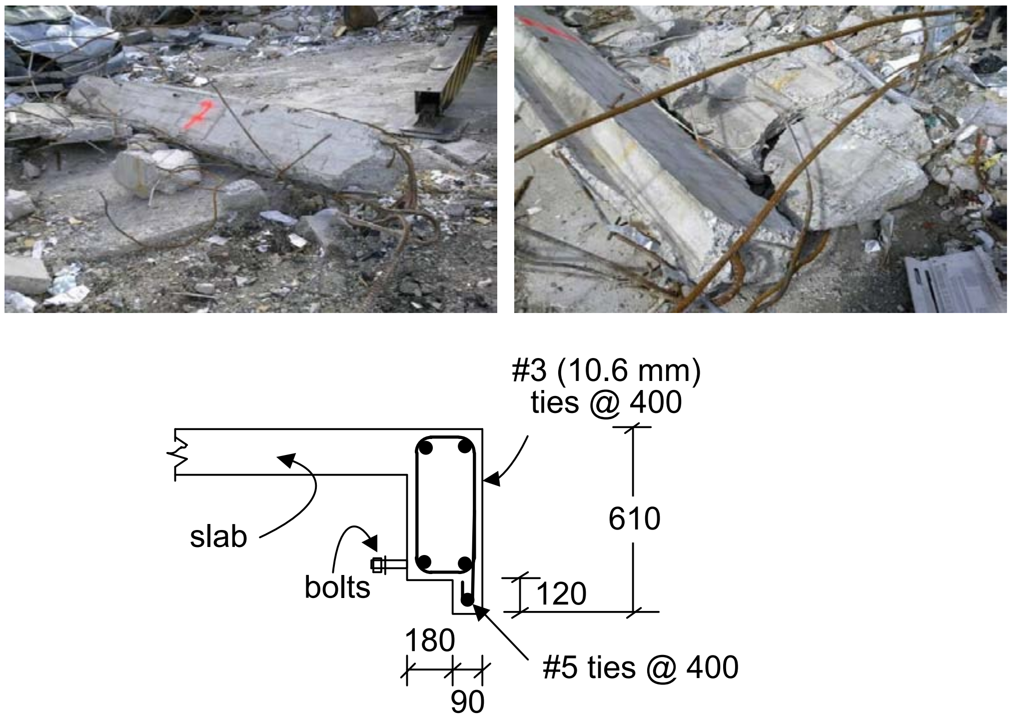

Figure 9 - Item 7 - 610 x 270 mm Concrete Spandrel Beam (photos clockwise from top left): a) Right hand end; b) Left end with slab remnant

H. ITEM 8A: 900 X 610 MM REINFORCED CONCRETE TO STEEL JACKETED COLUMN JOINT

This was a beam to beam and 410 mm diameter steel jacketed concrete column junction, similar to that in Item 2 (Figure 3). There were nine necked and fractured #7 bars seen on the underside of the junction (Figure 10).

Concrete had crushed and spalled away from the bottom four beam bars that ran through the junction.

The two transverse beams that ran into the junction each side had sheared off and stubs with diagonal fracture faced remnants were left.

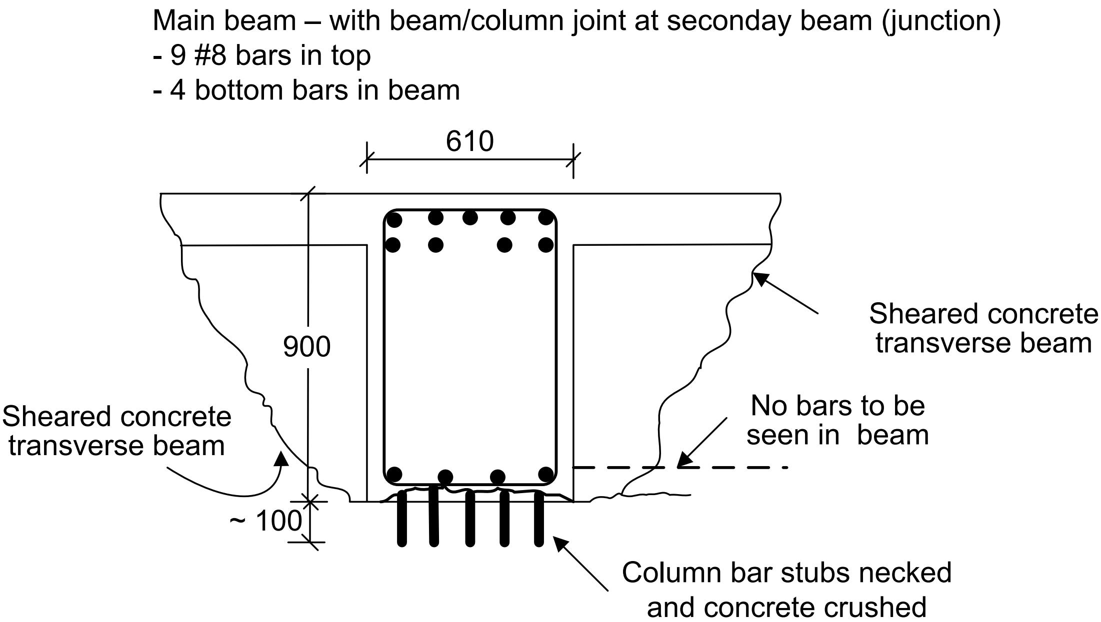

Figure 10 - Item 8A - 900 x 610 mm Concrete Beam to Steel Jacketed Column Joint (from left to right): (a) Underside of junction with 9 necked and fractured steel jacketed column bars and crushed concrete zone; (b) Underside of beam is on top with diagonal fracture face of tributary beam shown.

I. ITEM 11: 250 X 250 MM 8 BAR CONCRETE COLUMN'

Bars were extracted from this column for tensile testing by SAI Global (NZ) Ltd and chemical analysis by Pacific Steel Group.

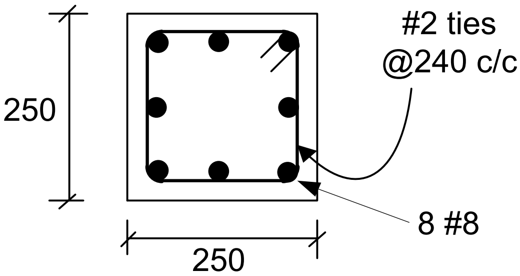

Figure 11 - Item 11 - 250 x 250 mm 8 bar Concrete Column with bars removed for tensile testing

J. SHEARED CONCRETE BEAM

Figure 12 - Sheared concrete beam attached to portion of floor still standing

K. CONCRETE ENCASED STEEL GIRDER ON STEEL JACKETED COLUMN

This was the end stub of a concrete encased double web welded steel girder on the portion of suspended floor still remaining standing (Figure 13).

The form of construction is consistent with that seen on the steel jacketed column with cap plate and fractured weld described in Item 3 (Figure 4).

Figure 13 - Concrete Encased Steel Girder on Steel Jacketed Concrete Column

3. REINFORCING STEEL PROPERTIES

A. TENSILE PROPERTIES

Reinforcing steel samples were extracted from items 1, 4, 6 and 11, then measured and tensile tested at SAI Global (NZ) Limited in Christchurch (Morris and Carson 2011). A copy of their test report P5675 is included in the appendices.

Tensile test results have been reported in accordance with the method of AS/NZS 4671:2001 (SNZ 2001). Nominal bar diameters used for determining stress values were the nominal imperial diameters described by bar numbers, converted into millimetres, and the metric nominal diameter for the Japanese D24 bars.

Deformation measurements were also reported.

A number of the #10 bars developed a rapid flat fracture post-yield, which initiated from notches perhaps caused by tack welding or thermal cutting during the initial construction (refer fig. 4 and 5 SAI Global Report P5675). The surface defect was encrusted with concrete residue and the bar had been removed by scabbling cover concrete and extracting using a cutting disc.

A summary of average properties measured for each bar size is shown in Table 1.

It is unknown whether the bars had been affected by in-service stresses that may have affected their tested properties.

Size	Uniform Elongation Agt (%)	Yield Stress Re: ReL or R0.2p (MPa	Ultimate Tensile Strength Rm (MPa)	Ratio Rm/Re	Comments
#4D	19.6	323	451	1.39	Items 6 and 16
#5D	18.5	293	411	1.41	Item 6
#8D	24.1	328	460	1.40	Item 11
D24	24.9	320	486	1.52	Item 1; Japanese metric; Bamboo deformations
#10D	12.5	424	696	1.64	Item 4; sample b excluded from Agt, Rm and Ratio due to post yield flat fracture

Table 1 - Reinforcing steel samples average tested properties

The current characteristic mechanical properties for 250N and 300E reinforcing are shown in Table 2.

The #4, #5, #8 bars conform to the requirements of NZS 1693:1962 (SNZ 1962) with minimum yield stress of 33 ksi (228 MPa); tensile strength between 55 and 75 ksi (380 to 517 MPa); and minimum elongation after fracture of 12% on 5 diameters gauge length. They also have tensile properties close to the lower bounds of AS/NZS 4671:2001 Grade 250N and 300E reinforcing (Table 2).

The #10 bars conform with NZS 1879:1964 HY60 ksi with minimum yield of 60 ksi (414 MPa) and tensile strength 1.2 x yield stress, but not less than 90 ksi (621 MPa); and minimum elongation after fracture of 12% on 5 diameters gauge length (SNZ 1964).

This achieved higher strength through the use of higher levels of carbon and other alloys resulting in a carbon equivalent that made it very susceptible to cracking when welded (Table 3). This can lead to sudden running flat fractures under load where notch defects are present, as was observed on site in Item 4 and during tensile testing.

Uniform Elongation Agt (%)	Yield Stress Re: ReL or R0.2p (MPa)	Ratio Rm/Re	Comments
≥ 5.0	≥ 250	≥ 1.08	250N AS/NZS 4671:2001
≥ 15.0	≥ 300 ≤ 380	≥ 1.15 ≤ 1.50	300E AS/NZS 4671:2001

Table 2 - AS/NZS 4671:2001 Characteristic mechanical properties limits for 250N and 300E reinforcing

B. CHEMICAL ANALYSIS

Reinforcing steel samples extracted from items 1, 4, 6 and 11 were sent to Pacific Steel Group laboratories in Otahuhu, Auckland for chemical analysis. The analyses were conducted using an ARL4460 Optical Emission Spectrometer following the ASTM E415 procedures. The carbon equivalent value WCE was calculated using the International Institute of Welding (IIW) carbon equivalent formula.

Pacific Steel analysis results are submitted to the Proficiency Test Program E-1, sponsored by the ASTM Committee E-1 (Analytical Chemistry for Metals). The results are set out in Table 3).

The D24 with bamboo deformations and "Japan" markings (PGC-Japan) had a chemical analysis comfortably conforming to NZSS 1693:1962.

The #10D (PGC-16311) HY60 had very high carbon equivalent (WCE) making it susceptible to running fractures, while conforming to the requirements of NZSS 1879:1964 of 0.40% w/w carbon (C), 0.050% phosphorus (P) and 0.050% sulphur (S). It would have been very difficult to weld reliably.

The #8D (PGC-11) bar was compared against the requirements for NZSS 1693:1962 which had no limits on Carbon, a 0.060% limit for Phosphorus and 0.060% for Sulphur. Its carbon equivalent is quite good. However the high levels of tin (Sn) and 0.066% phosphorus (P) meant that the steel would have had an elevated transition temperature and been more susceptible to running fracture from notch defects, particularly at low temperatures.

The #5D (PGC-6) bar conforms comfortably with NZS 1693:1962

Figure 14 - PGC bar samples after chemical analysis

Sample	C	Mn	Si	S	P	Al	Ni	Cr	Mo	Cu	Sn	V	WCE
PGC – JAPAN (D24)	0.19	0.45	0.08	0.017	0.013	0.010	0.03	0.04	0.007	0.08	0.016	0.002	0.280
PGC – 16311 (#10D)	0.41	1.26	0.16	0.029	0.019	0.012	0.07	0.05	0.007	0.16	0.018	0.002	0.646
PGC – 11 (#8D)	0.14	0.57	0.19	0.044	0.066	0.005	0.08	0.08	0.010	0.18	0.043	0.004	0.270
PGC – 6 (#5D)	0.13	0.41	0.14	0.032	0.010	0.014	0.06	0.04	0.008	0.15	0.016	0.001	0.219

NB: All figures are weight percentage values

Table 3 – Chemical analyses of reinforcing bar samples by Pacific Steel Group lboratory

4. CONCRETE PROPERTIES

Concrete cores were extracted on site from a 250 x 250 mm concrete column (Item1). This had external dimensions of 275 x 275 mm due to the existence of a solid plaster render. It has been reported as 275 x 275 mm in the concrete test report in Appendix B.

Concrete compressive testing was undertaken by Opus International Consultants Limited, Christchurch Laboratory (Jones 2011).

Concrete is widely known in the construction industry to strength-age or increase in strength over time. The amount of strength-aging is dependent on the mix design, batching, placement and curing practices. There is no quantitative relationship currently known for concrete manufactured in Christchurch. However the California Department of Transportation (Caltrans) found that in California concrete with 20 to 25 MPa specified 28 day strength had at least 25% strength –aging over 20 to 30 years. Concrete batching practice typically sought to achieve a target strength 20% greater than the specified 28 day cylinder compressive strength. This led to the use of a divisor of 1.5 on the strength-aged specimen test results to approximate the specified 28 day compressive strength (Priestley, Seible et al. 1996).

Average compressive strength from the three cores for the column was 47.3 MPa, with a minimum of 42.0 and maximum of 53.0 MPa. This is greater than the specified 28 day strengths for columns of both 3500 psi (24.1 MPa) and 4000 psi (27.5 MPa) specified, dependant on the column's location.

The expected column concrete test strengths accounting for grade variation and strength-aging were 36 MPa and 41 MPa respectively.

Concrete cores were also extracted from the 700 x 510 mm concrete beam (Item 5).

Average compressive strength from the three cores for the beam was 40.7 MPa, with a minimum of 38.5 and maximum of 42.0 MPa. This is greater than the specified 28 day strength for beams of 2500 psi (17.2 MPa).

The expected beam concrete test strength accounting for grade variation and strength-aging was 26 MPa.

The beam and column concrete tested therefore met the specified strengths at the time of the collapse. The tests also indicated that the concrete tested would have met the specified 28 day strength requirements at the time of construction.

5. CONCLUSIONS

A selection of structural remnants was extracted from the debris pile left after the collapse and de-construction of the building following the After-shock on 22nd February, 2011 and examined.

Mechanical properties of samples of the steel reinforcing and concrete were determined by testing.

The concrete compressive strengths measured were relatively high at over 40 MPa, but this is consistent with long term strength-aging over the estimated 47 years since construction and target batching practices. This indicates that the specified 28 day compression strength was approximately 32 MPa for the columns and 27 MPa for the beams tested.

The larger diameter #10 higher strength (HY60) bars found in deep beams were found from laboratory testing to be susceptible to running flat fracture prior to attaining their ultimate strength when surface defects were present.

Japanese D24 bars appear to have been substituted for #8 (25.4 mm) bars though their mechanical properties were found to be comparable with better chemical analysis than the #8 bars.

The #8 bars were found to have high levels of tin and phosphorus that would have elevated their transition temperature and made them more susceptible to running flat fracture than the Japanese bars.

Steel jacketed concrete columns were found that had developed necking and tensile fracture in their longitudinal reinforcing bars in conjunction with localised concrete compression induced bursting of the jacketing.

The cap of a steel jacketed concrete column was found to have the remains of a fractured 8 mm fillet weld that appeared to have connected the bottom flange of a 305 mm wide steel girder in similar form to others still remaining at Level 2.

Disclaimer:

The observations made in this report cover only a sample of structural remnants able to be accessed on the site at the time. They therefore need to be interpreted in conjunction with the original structural design drawings and specification, and modifications that may have occurred prior to the After-shock, as well as photos of the structure immediately after the After-shock and during its subsequent de-construction.

Some of the damage observed and documented may have occurred during deconstruction rather than as a direct result of the After-shock.

6. REFERENCES

Jones, G. (2011). PGC Building Concrete Drilled Core Tests Ref 5681-2. Christchurch, Opus International Consultants Christchurch Laboratory.

Morris, W. P. and A. L. Carson (2011). PGC Building Reinforcing Steel Tests P5675. Christchurch, SAI Global (NZ) Limited.

Priestley, M. J. N., F. Seible, et al. (1996). Seismic Design and Retrofit of Bridges, John Wiley.

SNZ (1962). Deformed Steel Bars of Structural Grade for Reinforced Concrete NZS1693:1962. Wellington, Standards Association of New Zealand.

SNZ (1964). Hot Rolled Steel Bars of HY60 Grade (60000 psi) for Reinforced Concrete NZS 1879 :1964. Wellington, Standards Association of New Zealand.

SNZ (2001). Steel Reinforcing Materials Standard AS/NZS 4671:2001. Wellington, Standards New Zealand.

APPENDIX A – REINFORCING STEEL TENSILE TEST RESULTS

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SAI GLOBAL

Date of Issue: 07 April 2011
Reference: P5675
Page 1 of 7 Pages

TEST REPORT

CUSTOMER: Hyland Consultants Ltd
P O Box 97282
Manukau
Auckland 2241

Attention: Dr Clark Hyland

CUSTOMER REFERENCE: Dr Clark Hyland – PGC Building

TEST SPECIFICATION: AS/NZS 4671:2001, Clause 7.2.2 (Tensile properties)
Steel reinforcing materials

AS 1391-2007
Metallic materials – Tensile testing at ambient temperature

ITEM TESTED: Three (3) #4R (12.7mm) reinforcing bar samples, (Unit4 16/3/11)
One (1) #4D (12.7mm) reinforcing bar samples, (6)
Three (3) #5D (15.9mm) reinforcing bar samples, (6)
Six (6) #8D (25.4mm) reinforcing bar samples, (5, 11)
Four (4) #10D (31.8mm) reinforcing bar samples, (unit4 16/3/11, beam unit4 16/3/11)
Three (3) D24 reinforcing bar samples (Bamboo)

DATE OF TEST: 21 March 2011

RESULTS: Refer to the body of this report.

The attention of the client is drawn to the statement of test policy annexed to this report, which form part of the terms of engagement between SAI Global (NZ) limited and the client.

Tested By:
W P Morris

Signatory:
A L Carson

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Imtest Group of Laboratories, part of SAI Global

Results of testing the mechanical properties of steel reinforcing to AS/NZS 4671:2001, Appendix C, Requirements for determining the mechanical and geometric properties of reinforcement

Synopsis

Various sizes of deformed reinforcing steel were supplied for testing to AS/NZS 4671:2001, Appendix C, Requirements for determining the mechanical and geometric properties of reinforcement.
Tensile tests were performed in accordance with AS1391 and percentage elongation measurements in accordance with ISO 15630-1.

The nominal bar diameters used for assessing the mechanical properties were the nominal imperial bar diameter converted to millimetres to one decimal place i.e. #5 =5/8" = 15.9 mm . With the exception of the three D24 "Bamboo" bars where the nominal metric size was used.

The sample markings on the reinforcing bars supplied are shown in figures 1 and 2. The #4D and #5D samples were marked with Pacific Steel and shown in figure 1. The #8D Bamboo samples were marked with JAPAN and are shown in figure 2. All the other samples had the same markings and are shown in figure 3.

Two of the #10D Unit 4 16/3/11 samples had sustained damage and are shown in figure 4 and figure 5.

C2 MECHANICAL PROPERTIES

C2.1 General

Tests for the determination of the mechanical properties of reinforcement shall be carried out at ambient temperatures in the range 10°C to 35°C.

The condition of test pieces at the time of testing shall be in accordance with Clause 7.2.1 and Table 3.

Unless otherwise specified, tests on bars and coils shall be carried out on straight test specimens of full cross-section having no machining within the gauge length.

Test specimens cut from mesh shall include at least one welded intersection. Before testing a twin-bar specimen, the bar not under test shall be removed with damage to the bar to be tested.

C2.2 Tensile properties

C2.2.1 *Equipment*

Tensile testing equipment shall be Grade A as defined in AS 2193.

C2.2.2 *Uniform elongation*

The uniform elongation (A_{gt}) shall be determined in accordance with ISO 15630-1 or ISO 15630-2 as appropriate except as in the following cases:

(a) All classes of steels – from extensometer measurements at maximum force taken during tensioning; or
(b) Class E and Class N steels only – from measurements taken after failure.

For the purpose of Item (a), a minimum extensometer gauge length of 50 mm may be used.
For the purpose of Item (b), gauge marks of up to 25 mm intervals may be used.
In the event of a dispute, the extensometer method shall take precedence, unless otherwise agreed between the parties concerned.

C3 GEOMETRIC PROPERTIES

C3.1 Rib geometry

C3.1.1 *Height of transverse ribs*

The height of transverse ribs (h) shall be measured for each row of ribs at the point where the rib height is greatest. The measurement shall be reported to an accuracy of 0.01 mm.

C3.1.2 *Circumferential spacing of transverse ribs*

The sum of the circumferential gaps (g) between adjacent rows of transverse ribs shall be measured at each of three separate cross-sections and the mean value of the sum calculated. The measurement shall be reported to an accuracy of 0.1 mm.

C3.1.3 *Longitudinal spacing of transverse ribs*

The spacing of the transverse ribs (c) shall be taken as the length of the measuring distance divided by the number of the rib gaps contained within that length. The measuring distance is deemed to be the interval between the centre-line of a rib and the centre-line of another rib on the same side of the product, determined in a straight line parallel to the longitudinal axis of the product. The length of the measuring distance shall contain at least 10 rib gaps.

C3.1.4 *Calculation of the specific projected rib area (f_R)*

The specific projected rib area (f_R) shall be calculated from the following equation, and with reference to Figure C1:
Note: The specific projected area was calculated in accordance with clause C3.1.4.

<u>Test Results</u>

Mechanical Properties

Sample Identification	Size	Measured diameter (mm)	Elongation at Maximum Force Agt (%)	Yield Stress Re, *Rp0.2 (MPa)	Ultimate Tensile Stress, Rm (MPa)	Ratio Rm/Re
6	#4D	12.16	16.4	*313	453	1.45
6a	#5D	15.32	16.2	290	409	1.41
6b	#5D	15.30	19.5	300	411	1.37
6c	#5D	15.25	19.7	289	414	1.44
5a	#8D	24.46	20.9	*306	464	1.51
5b	#8D	24.40	25.1	318	462	1.45
5c	#8D	24.20	24.9	313	451	1.44
11a	#8D	24.93	23.1	331	460	1.39
11b	#8D	24.80	22.5	329	460	1.40
11c	#8D	24.80	26.6	324	461	1.42
Unit4 16/3/11a	#10D	30.49	11.8	425	692	1.63
BeamUnit4 16/3/11a	#10D	30.65	12.4	418	696	1.67
Unit4 16/3/11b	#10D	30.71	4.0	423	652	1.54
BeamUnit4 16/3/11b	#10D	30.65	13.3	431	700	1.63
Bamboo a	D24	24.00	30.4	316	474	1.50
Bamboo b	D24	24.35	19.8	327	505	1.54
Bamboo b	D24	23.99	24.7	319	480	1.51
Unit4 16/3/11a	#4R	12.72	22.4	326	451	1.38
Unit4 16/3/11b	#4R	12.78	21.3	333	449	1.35
Unit4 16/3/11c	#4R	12.68	18.4	318	451	1.42

Table 1

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Geometric Properties (Not IANZ accredited)

Sample Identification	Size	Rib Height (h) (mm)		Circumferential gap (g) (mm)	Longitudinal Pitch (c) (mm)	Specific Projected Area f_R
6	#4D	1.00	0.77	0	6.1	0.14
6a	#5D	1.09	0.51	0	7.1	0.11
6b	#5D	1.01	0.50	0	7.2	0.10
6c	#5D	1.11	0.58	0	7.1	0.12
5a	#8D	1.05	1.10	0	11.9	0.09
5b	#8D	1.26	1.18	0	12.0	0.10
5c	#8D	1.23	1.14	0	11.9	0.10
11a	#8D	1.13	0.83	0	11.9	0.08
11b	#8D	0.87	0.83	0	11.9	0.07
11c	#8D	1.00	0.92	0	11.9	0.08
Unit4 16/03/11a	#10D	1.97	1.50	0	16.2	0.11
BeamUnit4 16/03/11a	#10D	1.91	1.83	0	16.2	0.12
Unit4 16/03/11b	#10D	1.64	1.72	0	16.2	0.10
BeamUnit4 16/3/11b	#10D	1.92	1.65	0	16.2	0.11
Bamboo a	D24	1.76	1.34	0	16.1	0.10
Bamboo b	D24	2.01	1.90	0	16.7	0.12
Bamboo b	D24	1.50	1.49	0	16.1	0.09

Table 2

Note: The circumferential gap is indicated as 0mm in all cases as the ribs extend for the entire circumference of the bar and intersect with the longitudinal ribs

Figure 1 – #4D and #5D

Figure 2 – D24 Bamboo

Figure 3 – All other bars

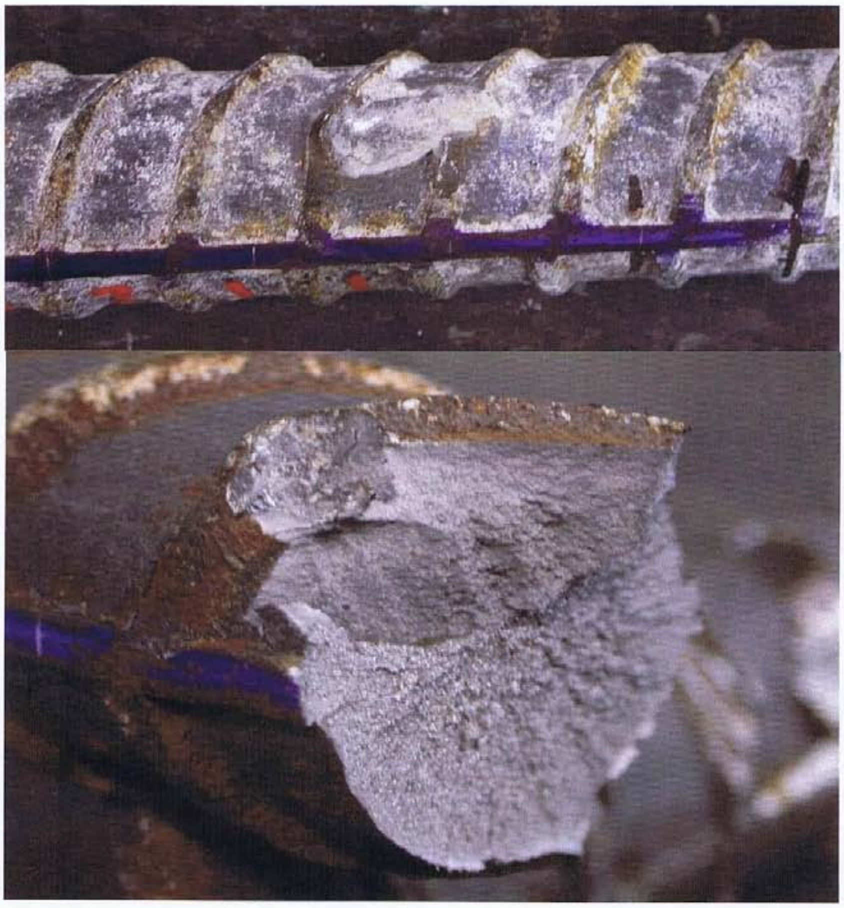

Figure 4 – Unit 4 16/3/11a before and after fracture

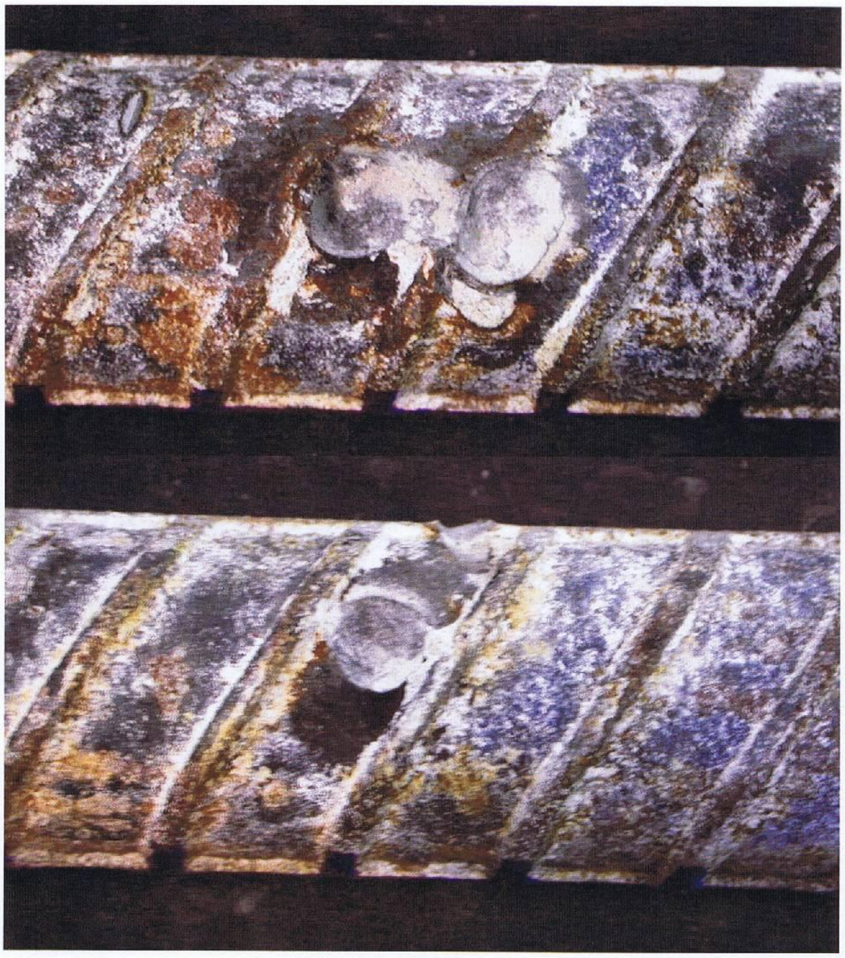

Figure 5 – Unit 4 16/3/11b

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APPENDIX B: DRILLED CONCRETE CORE TEST RESULTS

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CONCRETE COMPRESSION OF CORES TEST REPORT

Project :	Material Strength Investigation
Location :	PGC Building, Christchurch
Client :	Hyland Fatigue & Earthquake Engineering Limited
Contractor :	Concut Limited
Sampled by :	Concut Limited (John)
Date sampled :	16 March 2011
Sampling method :	Concrete Hole Saw (Horizontal)
Sample description :	Drilled Concrete Core
Sample condition :	Damp as received
Date cored :	16 March 2011
Source of concrete :	PGC Building, Beam 5
Grade of concrete :	Not Advised
Design strength :	Not Advised
Actual slump :	Not Advised
Date laid :	Not Advised

OPUS

Project No :	6-HFEE.11/006LC
Lab Ref No :	5682
Client Ref No :	Clark Hyland

Test Results

Lab reference no		067	067	067
Client reference no		PGC Beam 5	PGC Beam 5	PGC Beam 5
Date tested			28/03/11	
Dry cured	(days)		7	
Size & position of any reinforcement		No Steel	No Steel	No Steel
Visual description		Horizontal Core	Horizontal Core	Horizontal Core
Average core diameter	(mm)	92.8	92.4	92.8
Average core length	(mm)	183.8	184.4	185.5
Density	(kg/m^3)	2363	2382	2354
Height diameter ratio		1.98	2.00	2.00
Conditioning			Dry	
Load at failure	(kN)	283.8	257.1	282
Compressive strength	(MPa)	42.0	38.5	41.5
Type of fracture		Cone	Cone/Split	Cone/Shear

Test Methods	Notes
Testing of Cores, NZS 3112 : Part 2 : 1986, Clause 9 Compression, NZS 3112 : Part 2 : 1986, Clause 6 Density, NZS 3112 : Part 3 : 1986, Clause 5 Capping, NZS 3112 : Part 2 : 1986, Clause 4 (amendment No.2 2000)	Sampling is outside the laboratory's scope of accreditation

Date tested : 28 March 2011
Date reported : 29 March 2011

IANZ Approved Signatory

Designation : *Laboratory Manager*
Date : 29 March 2011

Sampling is not covered by IANZ Accreditation. Results apply only to sample tested.
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laboratory

Tests indicated as not accredited are outside the scope of the laboratory's accreditation

PF-LAB-095 (18/12/2010)

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CONCRETE COMPRESSION OF CORES
TEST REPORT

Project :	**Material Strength Investigation**
Location :	**PGC Building, Christchurch**
Client :	**Hyland Fatigue & Earthquake Engineering Limited**
Contractor :	**Concut Limited**
Sampled by :	**Concut Limited (John)**
Date sampled :	**16 March 2011**
Sampling method :	**Concrete Hole Saw (Horizontal)**
Sample description :	**Drilled Concrete Core**
Sample condition :	**Damp as received**
Date cored :	**16 March 2011**
Source of concrete :	**PGC Building, 275mm x 275mm Column 1**
Grade of concrete :	**Not Advised**
Design strength :	**Not Advised**
Actual slump :	**Not Advised**
Date laid :	**Not Advised**

OPUS

Project No :	6-HFEE.11/006LC
Lab Ref No :	5681
Client Ref No :	Clark Hyland

Test Results

Lab reference no		066	066	066
Client reference no		PGC Column 1	PGC Column 1	PGC Column 1
Date tested			28/03/11	
Dry cured	(days)		7	
Size & position of any reinforcement		No Steel	No Steel	No Steel
Visual description		Horizontal Core	Horizontal Core	Horizontal Core
Average core diameter	(mm)	67.6	67.4	67.6
Average core length	(mm)	137.1	139.0	139.0
Density	(kg/m^3)	2345	2352	2346
Height diameter ratio		2.03	2.06	2.06
Conditioning			Dry	
Load at failure	(kN)	150.3	188.6	168.1
Compressive strength	(MPa)	42.0	53.0	47.0
Type of fracture		Cone/Shear	Cone/Shear	Shear

Test Methods	Notes
Testing of Cores, NZS 3112 : Part 2 : 1986, Clause 9 Compression, NZS 3112 : Part 2 : 1986, Clause 6 Density, NZS 3112 : Part 3 : 1986, Clause 5 Capping, NZS 3112 : Part 2 : 1986, Clause 4 (amendment No.2 2000)	Sampling is outside the laboratory's scope of accreditation

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