Review of Concrete Testing and Interpretation in CTV Building Report: Site Examination and Materials Test Report done by Hyland Fatigue and Earthquake Engineering – 16th January 2012

James Mackechnie (PhD, CPEng) – April 2012

The Canterbury Earthquakes Royal Commission requested a peer review of concrete testing and interpretation reported in the Hyland Consultants report (BUI.MAD249.0190.5) prepared for the Department of Building and Housing [1]. The reviewed report entitled CTV Building: Site Examination and Materials Test Report was produced in January 2012 and summarises site and material findings taken from the CTV building from March 2011 onwards. The main report was also viewed for background details (BUI.MAD249.0189.1 – CTV Building Collapse Investigation; Hyland and Structure Smith) [2]. A brief review of other structural engineering investigations on the Royal Commission website was also undertaken.

1. Scope of peer review

The Hyland report is a wide ranging investigation that includes site examination and material testing of structural remnants of the CTV building. This peer review focuses on concrete technology aspects of the original report and in particular the strength of in situ concrete. I have read the Code of Conduct for Expert Witnesses and I agree to comply with it.

2. Concrete testing undertaken

All concrete tested in the Hyland report is summarised in Table 1 and 2 of this report, in terms of structural elements and concrete cores respectively. In virtually all cases, the structural elements were disturbed and exposed to varying degrees of damage from seismic stresses, fire, demolition and removal from site. This damage is evident from the photographic record and descriptions given in the original report. A subjective rating system for cracking risk based on this visual evidence is shown in Table 2.

The location where cores were extracted from these elements was documented in some cases and this is noted in Table 2. Schmidthammer testing of the surface hardness of concrete elements was done on most columns but the location of this testing along the element was not reported although this can be assumed from photographic records in the report.

3. Concrete testing methodology

Criticisms of the chosen methodology focus on the following issues; test element selection, core strength testing, Schmidthammer testing and microstructural assessment.

3.1 Test element selection

As stated in the preface of the Hyland report, findings were limited to the amount and quality of structural remnants available after an exhaustive rescue and recovery operation. A total of 29 concrete elements were tested, with twelve being tested directly for core strength while the reminder of columns had inferred strengths based on Schmidthammer results. Sampling appears to be representative in terms of assessing the range of damage but this was problematic in terms of trying to assess the original strength of concrete in the structure. For instance, only two Level 1 columns were clearly identified, namely tC2 and C18, but only column C18 was cored and tested. Column C18 was exposed to fire for several days and exhibited extensive cracking damage. In contrast, column tC2 was almost fully intact and was not near the fire centre or main part of the collapse. This column was likely to be more representative of the original concrete properties or would at least have produced more reliable in-place material properties had it been core tested.

Sampling of most columns was done transversely through elements, which limited the practical core diameter that could be extracted without intersecting steel reinforcement. Extracting cores with diameters less than 100 mm is not recommended when assessing heterogeneous materials such as concrete, since variability increases as core diameter reduces due to local defects having a more significant effect (cores should be at least four times the maximum aggregate size, e.g. 76mm for concrete containing 19mm coarse aggregate). These horizontal cores were also more prone to near-surface damage from flexure and fire. A more acceptable approach would have been to core down the centre of the column as was done on remnant of column C18 but without further mechanical breaking. This could have been done by diamond sawing columns into short lengths and extracting 100 mm diameter cores longitudinally.

3.2 Core strength testing

Core strength testing is covered in NZS3112 Part 2 and recommended practice in documenting testing is covered by CCANZ Information Bulletin 72 [3,4] . When conducting forensic investigations it is common practice to include a core log that documents the condition, location and orientation of the extracted material before sample preparation. The lack of the core log make interpretation of strengths difficult since intrinsic (material-related) issues such as compaction voids, bleed lenses and segregation cannot be determined and the location of material tested for strength is unknown.

Core strength reports undertaken by Opus International Consultants appear to be in accordance with the above guidelines but the quality and completeness of the information is adversely affected by the following:-

- mode of failure of core was often not reported
- mode of failure of core was sometimes unclear (e.g. column is not a failure mechanism)
- hardened density was not always reported (Lower Hutt laboratory report)
- location and orientation of intersected reinforcement in core was often not given

3.3 Schmidthammer testing

Schmidthammer testing was conducted on all columns except C18, which was noticeably fire damaged on the surface. A calibration curve was generated from the core strength results of the six columns tested and a relatively tenuous correlation was found as shown in Figure 61 of the Hyland report. Hammer strengths of the remaining 17 columns were then inferred from this relationship, which is acceptable in practice. The reliability of these hammer strengths is however influenced by the following unusual factors:

- surface deterioration caused by exposure to fire (no details are provided except for column C18)
- surface preparation of concrete before testing (photographs show a range of surfaces from off-shutter finish to heavily abraded with aggregates exposed)
- a calibration curve was developed across three different concrete mix designs where surface hardness would have differed due to varying levels of air entrainment and fine aggregate content

Hammer strengths become more variable with age and even when calibrated may range by ± 50% according to ASTM C805 [5]. It is unfortunate that more columns were not cored particularly since several almost fully intact columns were available for testing. Given the nature of the investigation, the most definitive set of testing results should have been generated.

3.4 Microstructural assessment

Assessment of concrete quality before testing was done by visual assessment alone and no microstructural assessment was undertaken on tested concrete. Cores extracted from the centre of column C18 were claimed to be unaffected by fire damage despite less than 100mm cover from the surface. A smouldering fire can produce temperatures in excess of 800 ^{0}C, which will raise internal temperatures above 500 ^{0}C after several days exposure. Significant strength loss is possible at temperatures above 300 ^{0} C and there is also the possibility of thermal shock due to fire fighting that can also cause cracking damage [6] .

Colour changes are often used diagnostically in situations were concrete is exposed to fire but there is no description provided in the report except for discussion on page 56 of the main report. Figure 56 shows a core and cored remnant from column C18 that exhibits a brown discolouration than is tentatively attributed to silt contamination. This is speculative given that the unlikely pattern of deposition on the fractured surface and the lack of any further supporting evidence. Several photographs (e.g. Fig. 48 and 49) showing columns exhibiting a pinkish, mottled surface, which is not discussed in the report and could come from a variety of sources, such as paint but could also have been due to fire damage. Concrete exposed temperatures between 300 to 600 ^{0}C can exhibit a pink colour from transformation of iron oxides within aggregates [7]. This discoloration converts to a grey/buff colour above 600 ^{0}C as shown on the surface of column C18 in Figure 47.

Cores were sometimes extracted close to fractures or macro-cracking and several cores were reported to have failed during coring. These highly stressed areas would have a high risk of cracking and could take the form of micro-cracking not seen with the naked eye. Microscopic examination of off-cut sections of cores by a petrographer should have been undertaken to establish the competence of the material before testing. A variety of diagnostic techniques are available to assess the crack density in concrete, including optical fluorescent microscopy [8].

4. Interpretation of concrete properties

Comments are made in four key areas of the interpretations presented in the Hyland report.

4.1 Core strength versus cylinder strength

The report does not address the issue of core strengths differing from cylinder strengths. Core strengths are generally lower than cylinder strength due to imperfections from core drilling and poorer curing and compaction of in situ concrete. These differences may be exacerbated when extracting smaller diameter cores, which have higher variability. Research shows that core strengths are typically 10-20% lower than cylinder strength made from the same concrete [9]. FEMA 274 recommends allowance be made for extrinsic (i.e. testing-related) factors including aspect ratio, core diameter, presence of rebar, core moisture content and damage from drilling [10]. No allowance was made in the Hyland report for core diameter, rebar or damage from drilling.

The use of rounded aggregates in the concrete also has an influence on core strengths since the inherently lower tensile strength will affect fracture toughness of concrete. Research indicates that core strengths of concrete containing rounded aggregate are generally lower than similar grade concrete made with crushed aggregates [11]. This difference may be exacerbated when extracting small diameter cores from structural elements.

4.2 Strength development with time

Concrete that is kept moist will carry on gaining strength over many years. The amount of long-term strength development is however dependent on the initial curing temperature such that precast concrete cured overnight at elevated temperatures might have had negligible long-term strength increase after 28 days. Similarly, relatively slender columns in the CTV building would have had limited volume to buffer external drying and the internal humidity of concrete would have quickly reduced to below the minimum level of 80% R.H. required for continued cement hydration. Long-term strength development gains of 25% that are used by Caltrans may be appropriate for larger bridge structures but beam, slab and column elements in CTV building were unlikely to achieve as much extra strength with time [12]. It is often assumed that long-term core strengths would not expected to be significantly higher than the initial grade strength [13].

4.3 Target strength at construction

Supply of concrete for the CTV Building was specified as being "High or Special Grade to NZS3104". The Hyland report assumes that High Grade was supplied for the project and uses target strengths based on this assumption. In contrast, all Christchurch concrete plants at the time had Special grading rather than High grading, with target strengths of 3 MPa lower than assumed in the Hyland report. Special grading was conferred to concrete plants with a proven testing regime where good technical control allowed the target to be set lower than for High grading and thus had lower margins between design and target strength . It should also be noted that these target strengths are used by concrete suppliers when testing concrete cylinders produced under ideal conditions and are not used in the structural design of the building.

4.4 Assessment of damage

Assessment of pre-existing damage in cores consisted of visual examination that was only able to identify the presence of macro-cracking. Similarly there was little attempt to quantify the risk of fire damage apart from visual and practical means (i.e. extracted cores were considered competent because they survived the core drilling operation and did not exhibit obvious signs of damage). It would be expected that more rigorous quantification would have been undertaken to assess the potential risk of significant micro-cracking damage. Such forensic investigation would have increased both cost and time but would have helped substantiate these findings and allowed more confident conclusions to be drawn. Petrographic examination of concrete would have provided valuable guidance and is often used in forensic investigations [14].

5. Missing construction information

The Hyland report makes no mention of the following information, which if available would greatly improve understanding of the concrete used in the structure:

- mix designs submitted by concrete ready-mix suppliers to the contractor and design engineer
- concrete batch records for each truck load of concrete supplied to project
- test certificates for concrete sampled at the concrete plant and on site
- any investigation reports about concrete issues during construction

Whilst the time elapsed makes it unlikely that detailed records would still be available, these data sources should have been investigated and any findings or omissions noted in the report. These data sources are a requirement for ready-mix concrete certified or audited by the New Zealand Readymix Concrete Association [15].

6. Overall analysis of findings

Determining the strength of concrete at construction is not a simple matter of applying a strength correction factor of 25% and requires a more rigorous methodology. Comparison of estimated strength with target strengths is further complicated by the Hyland report using High rather than Special grading. Strength results for column C18 are also assumed to be unreliable due to the high risk of fire and structural damage evident on the column stub and remnants. This is based on visual records, location within the collapsed building and proximity of the fire.

The average core strength of columns (except C18) was found to be 30.0 MPa while the average hammer strength was 27.5 MPa. This is consistent with a grade 25 MPa concrete, which is the mostly likely strength grade for tested columns. There is evidence of potentially lower strength elements such as column tR3, which had low density, core and hammer strength. Unfortunately gaps in reporting adversely affect the reliability of this data since the concrete quality is uncertain (i.e. was low density the result of compaction voids, higher capillary porosity, etc.) and the mode of failure was not reported (i.e. did cores fail normally indicating no compromising extrinsic factors).

Average strength of other concrete elements tested, namely slabs, walls and beams, was 29.0 MPa. Again there were elements with low strengths that were below the characteristic strength of 25 MPa but these had moderate to high risk of cracking based on proximity of cores to macro-cracking observed in these elements. Density of cores was more consistent and within the general range expected for grade 25 MPa concrete containing Christchurch greywacke aggregate. This would indicate that compaction and initial curing of concrete was adequate.

Analysis of core strengths done by the author found the following relationships (albeit with fairly poor correlations):

- strength increased with an increase in core diameter
- strength increased with reduced risk of cracking (based on the subjective visual rating in Table 2)
- embedded reinforcing steel did not have a consistent effect on core strength
- core strength increased with increasing hardened density of cores

7. Conclusions

Concrete material testing outlined in the Hyland materials report is lacking in terms of sampling and testing detail. While it is acknowledged that the building collapse, fire, demolition and relocation of structural elements made a systematic investigation challenging, the report draws from unnecessarily limited, incomplete and sometimes disparate sources. Criticisms of the testing process include selection of columns for coring, location of cores in highly stressed zones and lack of investigation of the potential for micro-cracking damage. Reporting of concrete testing was also sometimes incomplete or inconsistent, which limits the reliability of information.

The quality of information given in the report makes general conclusions difficult to draw, most notably for columns designed to have strengths of 35 or 30 MPa (Level 1 and Level 2 columns respectively). The conclusions drawn in the Hyland report about the likely strength of concrete are not fully supported by the evidence and ignore accepted guidelines used to interpret core strengths. Estimated strengths at construction are factored down by 25% without any reliable basis and then compared with target strengths that are 3 MPa higher than were required at the time.

The report is unable to distinguish between the residual core strength after the collapse and the likely in-place strength before 22nd February 2011. This is partly due to the limited amount of test samples available for testing, the choice and location of samples for core testing and the lack of quality assurance about possible seismic and/or structural damage of concrete cores used for testing.

The following can be reasonably stated based on the results from structural elements (not including column C18):

- average core strength of columns was 30 MPa
- average core strength of other elements was 29 MPa

Average core strengths reported were not much higher than the minimum specified value of 25 MPa and were lower than would be expected after 25 years had it been a well cured and relatively intact structure. Concrete strengths reported from other structures investigated by the Royal Commission were found to have had higher core strengths that significantly exceeded design strengths (tests were however conducted on larger, intact elements). This discrepancy found in the strength of concrete from the CTV Building was attributed by the Hyland report to the inherently low strength of concrete rather than to significantly greater risk of micro-structural damage. This claim is not substantiated from the information given in the report and a more rigorous investigation would be required to provide convincing proof of this opinion.

Table 1: CTV tested concrete element summary

Name of element	Geometry/ Element	Location within structure	Photograph number in HCL report	Visual condition of concrete based on photographic record	Subjective risk of cracking	Hammer strength (MPa)	Hardened Density (kg/m^3)	Compressive Strength (MPa)
tC1	400 ϕ Col.	Level 6	46f & 49a	Fully intact with damage at base	Low	29.8	2327	27.0
tC2	400 ϕ Col.	Level 1	54a	Fully intact with damage at base	Low	31.3	-	-
tC3	400 ϕ Col.	Unknown	53h	Short remnant with damage at ends	Mod.	29.0	-	-
tC4	400 ϕ Col.	Unknown	49b	Almost full length with end and middle damage	Low	44.3	2422	46.6
tC5	400 ϕ Col.	Level 6	49c	Full length with damage at base	Low	24.2	-	-
tC6	400 ϕ Col.	Level 6	53a	Short remnant with damage at lower end	Mod.	20.8	-	-
tC7	400 ϕ Col.	Unknown	53b	Short remnant with damage at both ends	Mod.	29.5	-	-
tC8	400 ϕ Col.	Level 5-6	53c	Short remnant, badly damaged at both ends	High	18.5	-	-
tC9	400 ϕ Col.	Level 6	53d	Half length, damage at base and middle	Mod.	23.2	-	-
tC10	400 ϕ Col.	Level 1-2	54b	Full length but in parts with surface damage	Mod.	18.4	-	-
tC11	400 ϕ Col.	Unknown	53e	Short remnant with multiple damage areas	High	19.4	-	-
tC12	400 ϕ Col.	Unknown	53f	Short remnant with major damage at ends	Mod.	33.2	2382	26.7
tC13	400 ϕ Col.	Unknown	53g	Short remnant with major damage at ends	Mod.	22.4	-	-
tR1	400x400 Col.	Unknown	55a	Half length with widespread damage	Mod.	23.4	-	-
tR2	400x400 Col.	Level 6	55b	Full length with damage along its length	Mod.	28.1	-	-
tR3	400x400 Col.	Unknown	55c	Half length with widespread damage	High	20.5	2247	20.3
tR4	400x400 Col.	Level 5-6	55d-f	Full length with localised edge damage	Low	37.0	-	-
tR5	400x400 Col.	Level 5-6	-	No photographic records	-	32.4	-	-
tR6	400x400 Col.	Unknown	56a	Short remnant with end and surface damage	Mod.	22.2	2387	25.5
tR7	400x400 Col.	Unknown	56b	Short remnant with widespread damage	Mod.	37.7	2351	40.9
tR8	400x400 Col.	Unknown	55g	Several remnant with damage along length	Mod.	28.1	-	-
tR9	400x400 Col.	Unknown	55h	Full length with end and middle damage	Low	23.4	-	-
tR10	400x400 Col.	Unknown	56c-d	Very short remnant with end damage	High	38.7	-	-
C18	400x400 Col.	Level 1	47	1m high remnant with fire damage & cracking	High	-	2335	16.0
E14	Floor slab	Line 4 Level ?	46b	Fractures locally and widespread cracking	Mod.	-	2365	27.3
E23	Hi-bond slab	Line 1 or 4 Level ?	46a	Diagonal cracking across slab	Mod.	-	2354	22.0
E4	South Wall	Level 4-5	46d	Cracking of unit but no visual cracks locally	Low	-	-	32.0
LW	North Core	Line 5 Level 1	46c	Fire damage on inside face of stair well	Mod.	-	2337	35.5
Logbeam	Precast beam	Line 2 or 3 Level ?	46e	Major damage next to core location	High	-	2330	25.0

Table 2: CTV tested concrete core summary

Core name Element – number	Element description	Diameter of core (mm)	Details of core as reported	Photo of core/coring in HCL report	Hardened Density (kg/m^3)	Core Strength (MPa)	Failure mode of concrete as reported
tC1-1	400 ϕ Col.	96	No steel	Fig. 46f	2324	26.5	Cone/shear
tC1-2	400 ϕ Col.	96	No steel	Fig. 53a	2331	16.0	Shear
tC1-3	400 ϕ Col.	96	19mm and 6mm bars	Fig. 53a	2443	27.5	Cone/shear
tC4-1	400 ϕ Col.	69	No steel	-	2412	47.8	Not established
tC4-2	400 ϕ Col.	69	No Steel	-	2433	45.3	Not established
tC12-1	400 ϕ Col.	69	No steel	-	2378	27.1	Not established
tC12-2	400 ϕ Col.	69	No steel	-	2385	26.2	Not established
tR3-1	400x400 Col.	69	No steel	-	2259	20.5	Not established
tR3-2	400x400 Col.	69	No steel	-	2234	20.1	Not established
tR6-1	400x400 Col.	69	No steel	-	2388	24.5	Not established
tR6-2	400x400 Col.	69	No steel	-	2385	26.4	Not established
tR7-1	400x400 Col.	69	No steel	-	2356	39.5	Not established
tR7-2	400x400 Col.	69	No steel	-	2347	42.2	Not established
C18-1.1	400x400 Col.	68	No steel	Fig. 47	2350	25.1	Not established
C18-1.2	400x400 Col.	68	No steel	Fig. 47	2330	12.8	Not established
C18-1.3	400x400 Col.	68	No steel	Fig. 47	2340	13.7	Not established
C18-2.1	400x400 Col.	69	No steel	Page 110	2310	16.5	Not established
C18-2.2	400x400 Col.	69	No steel	Page 112	2330	17.0	Not established
C18-2.3	400x400 Col.	69	No steel	Page 113	2360	11.0	Not established
E14-1	Floor slab	57	No steel	Fig. 46b	2359	25.0	Shear
E14-2	Floor slab	57	6mm bar	Fig. 46b	2380	30.5	Cone/split
E14-3	Floor slab	57	No steel	Fig. 46b	2355	26.5	Cone/split
E23-1	Hi-bond slab	57	6mm bar	Fig. 8b & 46a	2356	24.0	Shear
E23-2	Hi-bond slab	57	6mm bar	Fig. 8b & 46a	2347	22.5	Column
E23-3	Hi-bond slab	57	6mm bar	Fig. 8b & 46a	2358	19.5	Cone/split
E4(1)-A	South Wall	96	No details	Fig. 46d	-	30.0	Normal
E4(2)-A	South Wall	96	No details	Fig. 46d	-	33.0	Normal
E4(2)-B	South Wall	96	No details	Fig. 46d	-	31.0	Normal
E4(3)-A	South Wall	96	No details	Fig. 46d	-	34.0	Normal
LW-1	North Core Lift Wall	93	Horizontal D12 In top third	Fig 26c & 46c	2330	33.0	Shear
LW-2	North Core Lift Wall	93	Horizontal D12 in top third	Fig 26c & 46c	2350	34.0	Normal
LW-3	North Core Lift Wall	92	No details	Fig 26c & 46c	2330	39.5	Normal
Log beam	Precast beam	107	10mm bar	Fig. 46e	2330	25.0	Column

8. References

1. Hyland Fatigue + Earthquake Engineering, 'CTV Building: Site Examination and Materials Tests', Final Report (BUI.MAD249.0190.3), Department of Building and Housing, Auckland, January 2012.

2. Hyland, C. and Smith, A., 'CTV Building Collapse Report', Final report (BUI.MAD249.0189.3), Department of Building and Housing, Auckland, January 2012.

3. New Zealand Standards, 'NZS 3112 – Part 2 Methods of test for concrete – hardened properties', Wellington, 1986.

4. Cement and Concrete Association, 'Information Bulletin: Coring concrete for strength assessment', CCANZ IB72, Wellington, 2001.

5. American Society for Testing and Materials, ' Standard test method for rebound number of hardened concrete' ASTM C805-08, Philadelphia, 2008.

6. Mindess, S., Young, J.F. and Darwin, D., 'Concrete – Second Edition', Pearson Education, New Jersey, 2003.

7. Georgali, B. and Tsakiridis, P.E., 'Microstructure of fire-damaged concrete: A case study', Cement and Concrete Composites, 27, 2005, pp. 255-259.

8. Litorowicz, A., 'Identification and quantification of cracks in concrete by optical fluorescent microscopy', Cement and Concrete Research, 36, 2006, pp. 1508-1515.

9. Arioz, O., Tuncan, M., Ramyar, K. and Tuncan, A., 'A comparative study on the interpretation of concrete core strength results', Magazine of Concrete Research, 58, 2006, pp. 117-122.

10. Federal Emergency Management Agency, 'Seismic Rehabilitation Commentary' FEMA 274, Hyattsville, 1991.

11. Tuncan, M., Arioz, O., Ramyar, K. and Karasu, B., 'Assessing concrete strength by means of small diameter cores', Construction and Building Materials, 22, 2008, pp. 981-988.

12. Priestley, M.J.N. et al, 'Seismic Design and Retrofit of Bridges', John Wiley, 1996.

13. Neville, A.M., 'Properties of Concrete – Fourth Edition', Prentice Hall, 2000.

14. Broekmans, M.A.T.M., 'Petrography as an essential complementary method in forensic assessment of concrete deterioration; two case studies', Materials Characterisation, 60, 2009, pp. 644-654.

15. New Zealand Readymix Concrete Association, NZRMCA website – ww.nzrmca.org.nz

9. Declaration of affiliations

The author of this report in employed by Allied Concrete as its South Island Plant Engineer and has undertaken this role for the last four years. He is an adjunct senior fellow in the Department of Civil and Natural Resources Engineering at the University of Canterbury and is also a council member of the New Zealand Concrete Society.