CTV BUILDING

CTV BUILDING SITE EXAMINATION AND MATERIALS TESTS

FOR DEPARTMENT OF BUILDING AND HOUSING

16TH JANUARY 2012

Hyland

FATIGUE + EARTHQUAKE ENGINEERING

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REPORT PREPARED FOR:
DEPARTMENT OF BUILDING AND HOUSING

LIMITATIONS OF THE REPORT

During the rescue and recovery operation the building was largely deconstructed and the debris taken to the Burwood Landfill. This left a pile of structural remnants that had been set aside during the recovery operations due their apparent significance to the collapse. This report is therefore limited to observations about those structural remnants, the remaining tower and slab structure on site at the time, and column remnants extracted from the Burwood Landfill.

Structural remnants on the site were initially examined on 12 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 reinforced concrete columns, the collapsed Line 1 South Wall, the North Core lift and stair well walls by crane, and various beam and slab items.

Some of the damage shown in the photos and diagrams may have occurred during deconstruction and removal of debris. Where this is obvious it is noted.

The photos and diagrams therefore need to be interpreted in conjunction with the original structural design drawings and specification, and modifications that may have occurred prior to the Aftershock, as well as photos of the structure immediately after the Aftershock and during its subsequent de-construction.

EXECUTIVE SUMMARY

The CTV Building at 249 Madras Street (Figure 1) collapsed suddenly during the earthquake Aftershock on 22 February, 2011. Columns collapsed and floors fell on top of each other in a progressive collapse.

In the author's opinion the following factors, amongst other things, need to be considered in the analysis of the collapse of the CTV Building in the earthquake Aftershock on 22 February, 2011:

1. The concrete test results need to be interpreted recognising that the samples were extracted from components that had been damaged in the collapse. Care was taken however to avoid coring in or undertaking rebound hammer tests on portions of concrete with obvious cracks. Cored samples were visually scanned before testing for signs of cracking and conformity with the requirements of the testing standard NZS 3112:1986.

2. The minimum specified concrete 28-day strengths were 35 MPa for columns at Level 1; 30 MPa for columns at Level 2; and 25 MPa for columns from Level 3 to Level 6.

3. Based on the testing undertaken, it appears that at the time of the testing the column remnants from Levels 1 to 6 had mean concrete strength consistent with that of concrete with specified 28-day strength of 20 MPa.

4. Based on the testing undertaken it appears that at the time of testing the concrete in the South Wall and North Core may have met the minimum 28-day strength specified of 25 MPa.

5. Based on the testing undertaken it appears that at the time of the testing the concrete in the suspended slab may have met the minimum 28-day strength specified of 25 MPa.

6. A portion of reinforcing steel removed from the Line 1 South Wall near ground level appeared to have "work hardened" during the Aftershock and prior to the collapse of the building.

7. No evidence of settlement of the foundations and slab was able to be inferred from the site levels survey which found levels consistent with construction practice at the time of construction.

8. The north face of the Line 5 wall of the North Core was found to be out of plumb by an amount greater than the construction tolerances allowed in NZS 3109:1980.

9. Construction joints and interfaces between pre-cast components and other concrete elements were found to be typically smooth rather than roughened as is normally required to improve interface interlock.

10. Reinforcing steel from several pre-cast shell beams was not developed into the Line 4 core wall as specified.

11. Connection of the slabs by reinforcing steel into the Line D and D/E walls of the North Core was non-existent in some cases at Level 2, 3 and 4.

12. The connection of the C18 column (located at Line 4-D/E) into the lift core wall at Level 7 was less than specified and the bars had de-bonded.

13. A number of circular columns examined showed mid-height hinging failures as well as hinging at the base or head. This was also seen in a column remnant clearly identified as being a perimeter column that had been located between precast spandrel panels. Other circular columns were found full height with hinging damage at the head and base.

14. Rectangular columns which had all been located on Line A in the structure, typically exhibited beam-column joint failure as well as other damage.

Figure 1 3D Layout of columns and walls of the CTV Building

Disclaimer: This Executive Summary summarises the key points of this report and is not intended to be a substitute for the report in its entirety. The Executive Summary should be read in conjunction with the whole report and the reader should not act in reliance of the Executive Summary alone.

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 identifying reasons for the collapse of the building during the earthquake Aftershock on 22 February, 2011.

B. SCOPE

The Department of Building and Housing set the following scope for the investigation:

- Seek out relevant drawings of the structure.
- 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 CTV Building was located at 249 Madras Street. It was a reinforced concrete building with five suspended floor levels constructed with cast in-situ profiled metal deck and concrete floor slabs, precast concrete beams, circular concrete columns, and two sets of shear walls to laterally brace it (Figure 1).

A set of coupled shear walls ("the South Wall") was located on the Cashel Street or south end on Line 1 to which an external fire escape stair was attached. The other set of shear walls was located at the northern end ("the North Core") and was built around the lift and stair wells, and an amenities room.

According to records at the Christchurch City Council ("the Council") a building permit was granted on 30 September, 1986 for the construction of the building. Construction started in 1986 and finished in 1987 or 1988.

The building was severely damaged in the Aftershock on 22 February, 2011 and collapsed suddenly. A fire started in the stairwell area almost immediately and continued for some days.

The building was deconstructed down to the ground floor slab except for the majority of the line 4 lift core walls, by other parties. Items considered to be of structural significance were marked and set aside by others in a pile near the Cashel Street end of the site for examination. Another pile of general debris was located north of this area on a vacant lot.

In this report the Design Engineer is referenced using the abbreviation "DENG" and the Architect is referenced using the abbreviation "ARCH". Summary of Observations and Findings

A. PROFILED METAL DECK AND CONCRETE SUSPENDED SLAB

The profiled metal deck that formed the 200 mm thick slab had de-bonded from the underside of the concrete in some cases during the collapse. This is not unexpected as it is recognised by engineers that profiled metal decking does not rely on chemical adhesion with the concrete to develop the properties of composite profiled metal deck concrete slabs.

The steel decking had pulled away from the supporting beams in all cases except at the pre-cast beam support on Line 4 in front of the amenities room of the North Core. In that case the steel decking had torn away along the broken edge of the slab or a short distance in.

A portion of the decking was tensile tested and found to exceed the minimum specified yield stress of 550 MPa (p.17)

B. PRE-CAST CONCRETE SHELL BEAMS

The pre-cast concrete shell beams were found to have no reinforcement in the in-situ in fill concrete. However this was consistent with the how they were specified.

There was no roughening of the precast surface on the inside of the shell beams to encourage composite action between the shell and the in-fill concrete (p19).

The slab on the shell beam on Line 4 that connected into the shear core wall had fractured along the inside edge of the beam.

The bottom reinforcing steel in the shell beams had not been developed fully into the Grid C core wall on Line 4 as specified except at Level 2. The bars had been bent back into the concrete infill in the shell beam (Figure 7).

C. 400 MM DIAMETER COLUMNS

The exterior 400 mm diameter column (Item E33) had flexural failure at the floor level lap joint of the vertical reinforcing steel and compression-flexural fracture at the upper end of the column (Figure 10)

The lap joint in the exterior columns was concealed by the external spandrel panels and interior linings (Figure 72 and Figure 73).

D. INTERNAL PRE-CAST LOG BEAMS ON LINE 2 AND 3

The ends of the pre-cast internal log beams that supported the 200 mm thick profiled metal deck slab had smooth formed un-roughened ends at the interface with the beam–column joint zone. This would have reduced beam-column joint shear capacity (Figure 12).

E. EXTERNAL PRE-CAST LOG BEAM ON LINE 1 AND 4

The ends of the pre-cast log beams supported by the corner columns on Grid A had a smooth un-roughened end where it connected into the columns reducing the beam-column joint shear capacity.

No starter bars connected the log beam into the 200 mm slab that was supported on the shell beams (Figure 13).

F. SOUTH WALL ON LINE 1

The Line 1 South Wall that extended from Level 1 on the ground to the roof had been broken up into single story components during de-construction (Figure 74).

i. Level 1 to 2 (Item E1)

This panel showed flexural cracking patterns typical of cantilever shear walls (Figure 14).

Reinforcing steel taken from the east end of the wall was found to have yielded and elongated prior to the collapse of the building (page 55).

ii. Level 2 to 3 (Item E2)

This panel had diagonal cracking in the piers consistent with cantilever wall behaviour and two way diagonal cracking in eth door head coupling beam (Figure 16).

iii. Level 3 to 4 (Item E3)

This panel had dominant uni-directional diagonal cracking running from the bottom west corner to the top east end (Figure 18).

Severe crushing damage had occurred at the junction of the wall with the attached pre-cast shell beam B15 at level 4 that ran to the Grid F/1 column (Figure 65).

iv. Level 4 to 5 (Item E4)

Severe two-way diagonal shear cracking in east pier and loss of cover to vertical reinforcing steel on east edge.

Smooth mortar construction joints rather than roughened at junctions with pre-cast shell beams B15 and B16 (Figure 67).

v. Level 5 to 6 (Item E5)

Weak concrete in west pier adjacent to top of doorway that was able to be dislodged by boot (Figure 22).

The top surface of wall was smooth rather than a roughened construction joint for slab seating.

Bars from wall into attached pre-cast beam had fractured.

No obvious cracking had occurred in the wall or the door head coupling beam.

vi. Level 6 to Roof (Item E5A)

No obvious cracking had occurred in the wall piers or door head coupling beam (Figure 24).

G. NORTH CORE WALLS

Horizontal flexural cracking on west and north face at Grid C/5 (Figure 25).

Fine two-way diagonal cracking on the inside faces of Level 1 to 2 walls (Figure 26).

H. SLAB AND BEAM REMNANTS ON LINE 4 OF NORTH CORE

The extent of the slabs at the time of examination was measured (Figure 32).

Portions of the level 6 and Level 5 slabs that were still attached immediately after the Aftershock were removed during deconstruction for safety reasons. The slab at level 2 had also been broken back. The rest of the slab was in the condition it was left after the event.

i. Level 6 Slab

The slab had a vertical fracture face that coincided with the ends of the H12 saddle bars from the support beam on Line 4 (Figure 27).

664 mesh in the slab had fractured in a ductile manner.

The profiled metal deck steel decking had fractured in tension adjacent to the edge of the fractured slab edge.

The imprint of the bent back bottom bars from the pre-cast shell beams (Figure 7) was visible in the cover concrete of the wall (Figure 2).

ii. Level 5 Slab

The fractured edge of the slab was similar to that at level 6.

Reinforcing was located in the bottom of the slab rather than as specified near the top surface (Figure 28).

The imprint of the bent back bottom bars from the pre-cast shell beams (Figure 7) was visible in the cover concrete of the wall (Figure 28).

Cracks were found running from cores drilled in the slab for pipes.

iii. Level 4 Slab

The imprint of the bent back bottom bars from the pre-cast shell beams (Figure 7) was visible in the cover concrete of the wall (Figure 29).

The profiled metal deck decking of the fractured slab was still clamped to the support beam on Line 4 and fractured in tension.

iv. Level 3 Slab

Similar to Level 4

v. Level 2 Slab

Bottom bars of pre-cast shell beam had been developed into the core wall on this level and beam-column joint type diagonal cracking was seen on the end of the wall consistent with cyclic demands having occurred during the Aftershock.

Figure 2 - West face of North Core (Wall Line C) showing the locations where the Line 4 beams had connected into the wall at Levels 3 to 6 without the bottom bars being developed into the wall as specified.

I. SLAB CONNECTIONS TO NORTH CORE ON GRID D AND D/E

Drag Bars had been fixed into the slab and into the shear walls at Levels 4, 5 and 6 with epoxy grouted threaded anchors after the original construction had been completed (Figure 33).

i. Level 2 Connection of Slab to Walls D and D/E

No reinforcing steel connected the slab to the east wing wall D/E.

A 20 mm hole was found in the west wing wall D where a reinforcing bar had pulled out (Figure 34).

ii. Level 3 Connection of Slab to Walls D and D/E

An H12 bar was found fractured at the end of the west wall D.

No reinforcing steel was found to have connected the east wing wall D/E to the slab (Figure 35).

iii. Level 4 Connection of Slab to Walls D and D/E

The Drag Bars on both the west and east wing walls had partially fractured in bending and tension. The epoxy grouted 20 mm threaded anchors that were fixed vertically into the slab and into the Drag Bar on the west wall appeared to have pulled out in tension as the slab pried it off as it rotated downwards during the collapse (Figure 36 and Figure 37).

The 20 mm diameter Drag Bar threaded anchor rods were hardness tested by MTL (Figure 62) and found to have Rockwell Hardness HRB greater than the minimum required by AS 4291.1:2000 (SAA 2000) for Property Class 5.8 threaded rods.

iv. Level 5 and 6 Connection of Slab to Walls D and D/E

Similar to what was seen at Level 4 (Figure 39 and Figure 40).

J. CONNECTION OF COLUMN C18 TO NORTH CORE AT LEVEL 7

The column had pulled away in tension from the connection at the North Core wall D/E. Three 20 to 24 mm diameter holes were visible where bars connecting the C18 column had pulled out. Four H20 bars were specified on the drawings to be developed into the wall (Figure 41).

K. LEVELS AND POSITIONAL SURVEY

The floor slab, slab overlay and foundation beams were found to have levels consistent with original construction tolerances and practice.

No evidence of long term foundation settlement or settlement induced by the Aftershock could therefore be inferred.

The North Core Line 5 wall was surveyed for verticality by sighting on the eastern and western corners of the north face of the wall. It was found that there was a northwards out-of-vertical measurement of 91 mm over 18.53 m between Level 1 and Level 7 at the northeast corner, and 68 mm over 18.53 m at the northwest corner (Figure 57).

This is much greater than the straightness limit of 30 mm for structures greater than 10m high or position plan tolerance of 10 mm in NZS 3109.

OTIS, the company that maintained the lifts at the CTV building, advised that they had no records of the inside faces of the walls being out-of-plumb after construction to an extent that it affected the installation of the lifts. No damage was found to the foundation beams around the core (Figure 57).

L. REINFORCING STEEL PROPERTIES

Reinforcing steel samples were extracted from the Line 1 South Wall and tested to determine tensile properties, production uniformity and work hardening during the Aftershock.

664 mesh from the suspended slab was also sampled and tested.

The reinforcing steel was found to conform to the standards of the day.

The H28 steel extracted from the lower portion of the Line 1 South Wall E1 was found to have elongated 3.3 % more than the other 16 to 28 mm bars extracted. It also had an elevated yield stress. This indicated that the bar had work-hardened during the Aftershock and prior to the collapse of the building (Table 1).

The chemical analysis of the 16 to 28 mm bars found that they had chemical compositions consistent with them being from the same of similar production runs (Table 2).

M. CONCRETE PROPERTIES

Cores were extracted from columns, beams, slabs and walls for compressive strength testing (Figure 46). The chord modulus of elasticity was also determined for the South Wall and North Core concrete.

The sample means of the test results for a particular member were assessed against the known means of concrete properties with 28-day strengths conforming with NZS 3104:1983.

A lower 0.1% acceptance limit was applied to identify upper bound conformity with a specific strength category. Where the sample size was sufficiently large an upper

0.1% rejection limit was also applied to identify non-conformity with a lower strength category.

The 28-day strength of concrete is used to identify a lower bound strength (close to the lower 5 percentile strength) that could be expected from concrete made to comply with the Specification for Concrete Production NZS 3104:1983. The 28-day strength is typically used by structural engineers when calculating the design capacity of concrete members.

The concrete test results need to be interpreted in light of the fact that the samples were extracted from components that had been damaged in the collapse. Care was taken however to avoid coring in portions of concrete with obvious cracks and samples were visually scanned before testing for signs of cracking for conformance to the requirements of the concrete testing standard NZS 3112:1986.

The suspended slab presented the greatest difficulty in achieving this due to the condition of the remnants accessible for testing.

i. Suspended Slab Concrete Properties

The suspended slab concrete was core tested in two locations. The average strength at test was 24.6 MPa.

This indicates that the concrete had strength not greater than that with 28-day strength of 25 MPa.

The specified 28-day strength was 25 MPa.

ii. South Wall and North Core Concrete Properties

Concrete cores extracted from one location each in the South Wall and the North Core found an average strength of the walls of 33.8 MPa. This became 36.5 MPa when adjusted 8% for testing orientation transverse to casting direction.

This indicates that the concrete had strength not greater than that with 28-day strength of 35 MPa.

The specified 28-day strength was 25 MPa.

The chord modulus of elasticity of the shear wall concrete was found to be an average of 27,600 MPa.

The calculated average secant modulus of elasticity was 26,100 MPa (page 64).

iii. Column Concrete Properties Summary

The core and rebound hammer tests, indicated that the strength of the column remnants as a whole, based on the testing of 26 column remnants selected at random from the debris, had a mean strength at the time of the collapse of 27.4 MPa. This increased to 29.6 MPa when adjusted 8% for testing orientation transverse to the direction of casting (Table 4).

This indicates that the concrete had strength consistent with that of concrete with 28-day strength of 20 MPa. This is less than the least concrete column 28-day strength that was specified of 25 MPa.

2. EXAMINATION OF STRUCTURAL REMNANTS

The examination of structural remnants was undertaken by the author and a DBH Engineer, on Saturday 12 March, 2011 (Figure 3). It was then visited again with an engineer from Structuresmith Ltd on 5 April, 2011.

Observations and comments are recorded about each item in the general text and in captions to the photos.

Figure 3 – Structural debris pile on CTV site (top to bottom) (a) At start of Site Examination; (b) Crane used to move debris remnants for examination

A. FOUNDATIONS AND GROUND FLOOR SLAB ON GRADE

The ground floor slab had a concrete overlay that measured on average 89 mm thick over the eastern half of the floor (Table 5). This ramped up from the original slab adjacent to the lift core (Figure 4).

The slab appeared to be in reasonable condition and there weren't any obvious heave or localised damage at column or shear wall locations. A "levels and positional survey" was undertaken to check for signs of settlement and lift core rotation and is reported in Section 3.

All the concrete columns had been removed to floor slab level except for a 400 mm square column stub C18 stub adjacent to the east end of the lift core walls (Figure 47).

Figure 4 - Ground floor Level 1 slab on grade (clockwise from top) (a) View from snorkel of debris and western portion of Level 1 slab on grade; (b) Slab in northwest corner with column reinforcing protruding; (c) Ramp formed in concrete overlay in front of Line 4 lift core walls.

B. PROFILED METAL DECK AND CONCRETE SUSPENDED FLOOR SLABS

The drawings specified a 200 mm thick profiled metal deck floor slab spanning north to south and seated on 400 mm wide precast log beams at 7500 mm centres (DENG Dwg S15(Figure 65)). The slab was specified to be propped during construction with the trays pre-set upwards at approximately quarter points to a maximum of 20 mm at midspan and then the topping (Clause 2.16 Figure 77) cast to provide the specified thickness.

Reinforcing mesh size 664 was specified under H12 saddle bars 4000 mm long at the internal beams on Lines 2 and 3, and draped to 20 mm above the profiled metal deck at midspan.

The Base Metal Thickness (BMT) of the profiled metal deck was not stated on the Drawings but is called up as grade G500 with 0.75 BMT in the Specification. A sample taken on site was measured by SAI Global Ltd testing laboratories to have a mean thickness including galvanising of 0.81 mm indicating that 0.75 BMT Hi-Bond had been used. The average tensile strength of the sheet was measured to be 617 MPa (refer page 105).

The profiled metal deck decking remnants observed were typically found to have de-bonded from the concrete topping. This is consistent with the way metal decking behaves in composite floor slabs. The rib interlock and interface friction between the concrete and steel sheet being the principal means of developing shear flow between the steel deck and the concrete topping.

The decking had remained clamped between the slab and the supporting precast beam on Line 4 at the lift core (ARCL B24 Dwg S18 (Figure 67)) as seen at Level 4 in Figure 29. The decking had fractured during the collapse.

The decking and slab had pulled away from the adjacent edge beams to the west of the lift core on Line 4 (DENG B22 and B23 Dwg S18 (Figure 67) as seen in Items E14 (Figure 7) and E18 (Figure 13). On the edge beam Item E23 (Figure 8) the decking had pulled away from under the portion of remaining slab cantilevering from it.

The slab had pulled away completely from the interior pre-cast log beams from Lines 2 and 3 (DENG B1 to B10 Dwg S18 (Figure 67) and Section 8 Dwg S15 (Figure 67)), as seen in Figure 11 and Figure 12.

C. ITEM E21: ARCHITECTURAL CLADDING PANEL

Figure 5 - Pre-cast spandrel panel Item E21 (DENG Dwg S25 (Figure 65))

D. SHELL BEAM AND SLAB

i. Item E6

Figure 6 - Edge shell beam Item E6 showing unreinforced concrete infill and smooth interface between shell beam and in-fill. The DENG Specification Precast Concrete cl 3.12 required roughened interface surfaces (Figure 77).

ii. Grid 4 / B-C; Item E14

Figure 7 - Pre-cast shell beam (Item E14) from northern face Grid 4, west side of lift core (DENG B23 Dwg S18 (Figure 67)). (clockwise from top left) (a) Top face with slab fracture along edge of shell beam, extending out further at far end adjacent to lift core attachment; (b) to (d) Fractured slab outstand remnant at east end from which slab concrete cores were extracted. The bottom H24 bars from shell beam have been turned back into the concrete infill rather than embedded in shear wall as specified (DENG Detail 5 Dwg S19). Refer also bar imprint on wall at the connection seen in Figure 29 at Level 4 and Figure 30 at Level 3.

iii. Item E23

Figure 8 - Edge Shell Beam (Item E23) from Line 1 or 5. (Clockwise from top left) (a) Underside and outer face; (b) Underside showing 1200 mm slab outstand with metal decking pulled away and diagonal cracking indicating shear in diaphragm; Holes are where concrete cores were taken for testing; (c) Carpet remnant on top of slab; (d) Damaged shell beam.

E. 400MM DIAMETER CONCRETE COLUMNS

i. Item E19

Figure 9 - 400 mm diameter column Item E19. (Left to right) a) Level 6 to Roof, likely location Grid F (Figure 65) based on roof steelwork hold down attachment detail; b) Flexural fracture at base in lap zone of vertical reinforcing steel. R6 spirals at 250 centres can be seen (Figure 69).

ii. Item E33

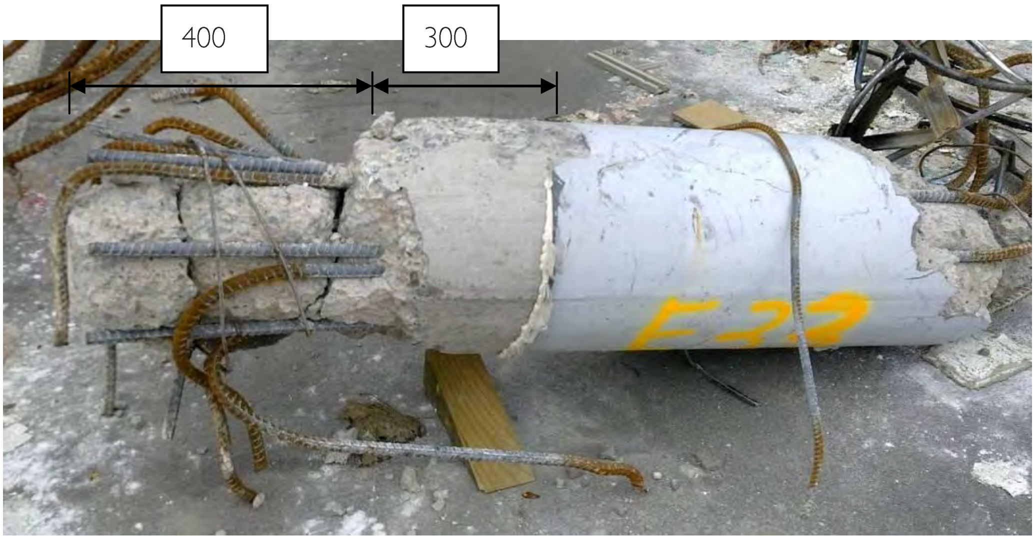

Figure 10 - 400 Diameter Exterior Column Item E33. (DENG C5 or C11, Dwg S15 (Figure 69)). Left end is bottom of column at floor level with concrete spalling over lapped vertical reinforcing. Horizontal cracking could be seen in the core confined by R6 spiral which had fractured. The unpainted portion measured at 700 mm long indicates it had been located between spandrel panels (Figure 5, Figure 72 and Figure 73). The fracture at the right end of the picture commenced approximately 1350 mm from the base of the column.

F. LINE 2 AND 3 INTERNAL PRE-CAST LOG BEAMS

i. Item E26

Figure 11 - Interior Pre-cast Log Beam from Line 2 and 3 (DENG Section 3 Dwg S15 (Figure 66)) (left to right) (a) Diagonal shear damage at end and smooth formed surface at beam-column joint; (b) Mid-portion of beam concrete has broken away and stirrups are pulled apart.

ii. Other Log Beams

Figure 12 - Interior Pre-cast Log Beams from Line 2 and 3 (DENG Section 3 Dwg S15 (Figure 66)) showing smooth concrete formed for beam-column joint and bottom hooked bars that have pulled out of beam-column joints without any obvious straightening; metal decking has pulled away from slab seating; no slab remains attached to the beams

G. LINE 1 AND 4 EDGE PRECAST LOG BEAM: ITEM E18

Figure 13 - Item E18 Pre-cast edge beam north-west corner (DENG B22 Dwg S18 (Figure 67) (from left to right) (a) Smooth form finish at attachment to column 4A (DENG Detail 1 Dwg S19 (Figure 70)); (b) No starter bar reinforcing steel extended from this pre-cast beam into slab. This was consistent with what was shown on the Drawings (DENG Section 4 Dwg S15 (Figure 66).

H. LINE 1 SOUTH WALL

The Line 1 South wall ran full height from Level 1 to the Roof. During deconstruction the wall was broken into six floor to floor portions and labelled E1 to E5A. The number refers to the level at the bottom of the wall portion, except for E5A which was located on Level 6 (Figure 74).

The relevant damage and features are noted in the photos captions and shown diagrammatically in the associated sketches.

i. <u>Line 1 South Wall Level 1 to 2: Item E1</u>

Figure 14 - Line Shear Wall (Item E1) (clockwise from top left) (a) Outer face of wall with lower portion of concrete removed during deconstruction exposing the reinforcing steel; (b) Outer face with cracks highlighted by red paint; (c) Inside face with cracks highlighted by red paint; (d) Top west corner; (e) Top east corner; (f) Inside face of east pier

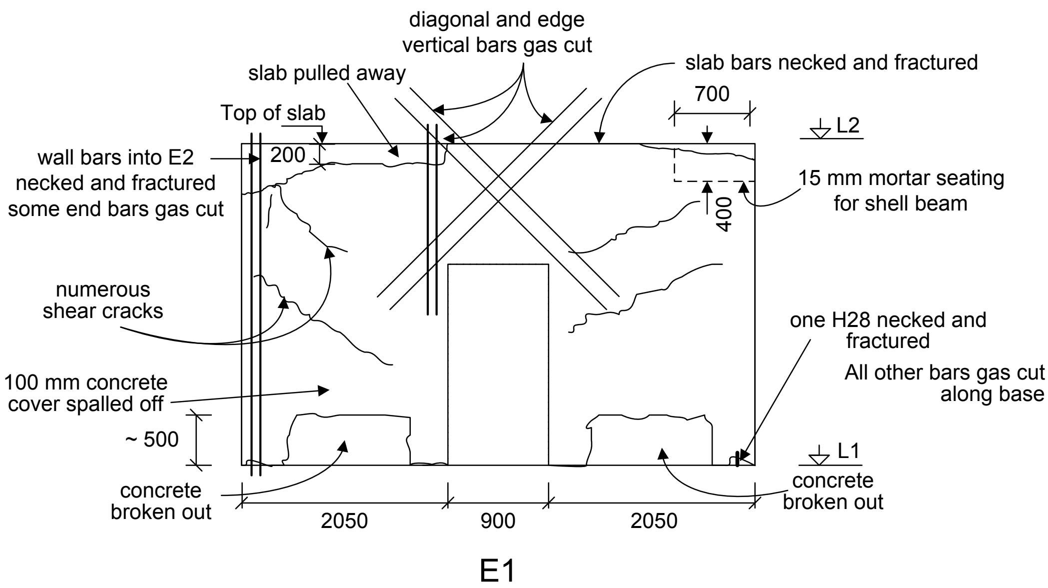

Figure 15 – South Wall Item E1 schematic of damage observed.

ii. Line 1 South Wall Level 2 to 3: Item E2

Figure 16 - Line 1 South Wall Level 2 to Level 3 (Item E2) (clockwise from top left) (a) Outside face with fire escape door attached. Cracks marked by red paint; (b) Inside face of wall; (c) East pier construction joint with necked and fractured bars indicated by red paint; (d) Escape door edge of east pier showing thick cover concrete to reinforcing; (e) Outer edge of west pier showing necked and fractured bar indicated by red paint others were cut; (f) Outer face of wall with cracks and fractured bars marked.

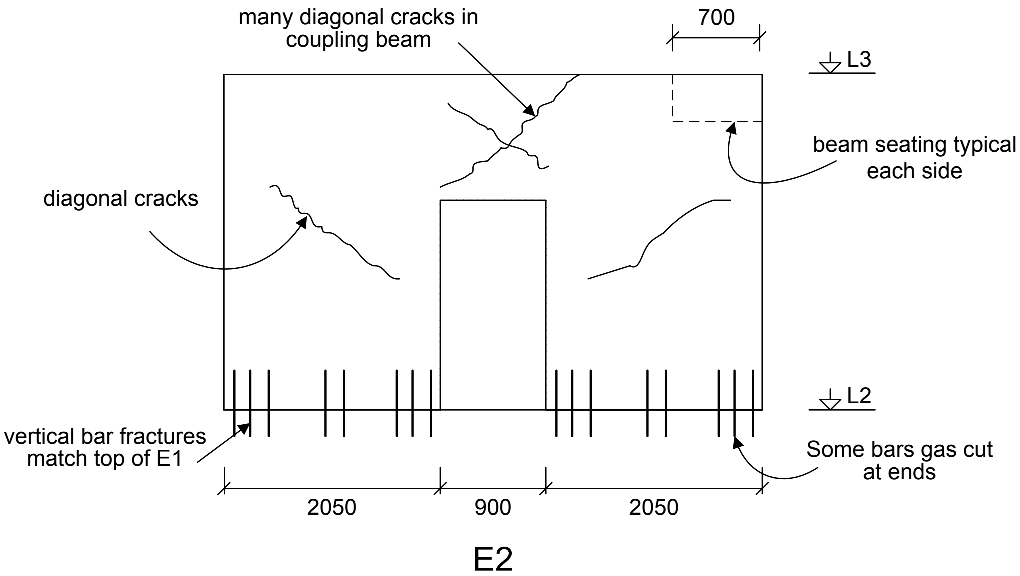

Figure 17 - South Wall Item E2 schematic of damage observed.

iii. Line 1 South Wall Level 3 to 4: Item E3

Figure 18 - Line 1 South Wall Level 3 to Level 4 (Item E3) (clockwise from top left) (a) Outer face; (b) Inner face; (c) Damaged top east corner; (d) One way diagonal cracks running from bottom west to top east side marked by paint on outer face.

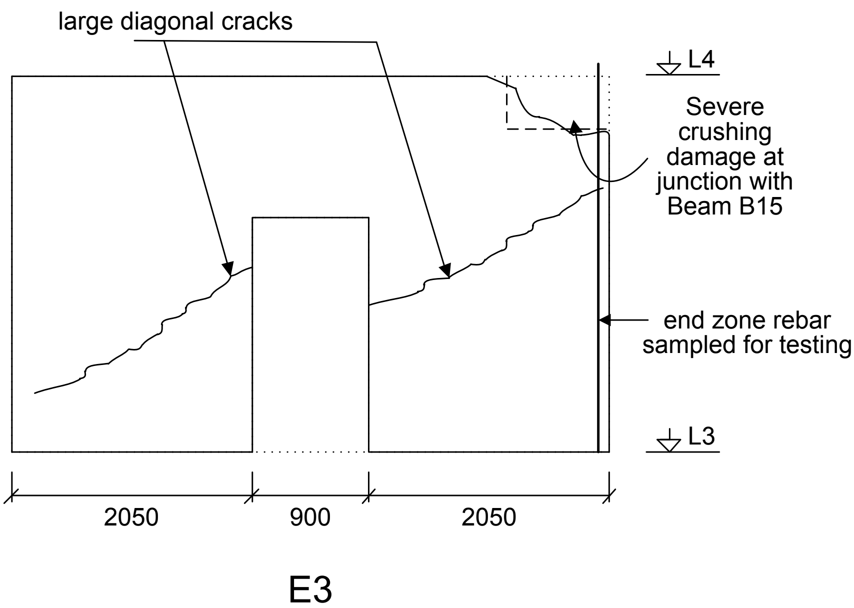

Figure 19 - South Wall Item E3 schematic of damage observed.

Figure 20 - Line 1 South Wall Level 4 to Level 5 (Item E4) (clockwise from top left) (a) Outer face with east pier on right with severe shear damage, and timber formwork remnant; (b) and (c) Charring on fractured concrete surfaces prior to deconstruction; (d) Top west corner showing saw-cut on top edge from deconstruction; (e) Top east corner showing smooth construction joint at interface with pre-cast beam B15 (DENG Dwg S18 (Figure 67)) and fire charring to spalled eastern edge; (f) View from east to west of top east corner construction joint notch.

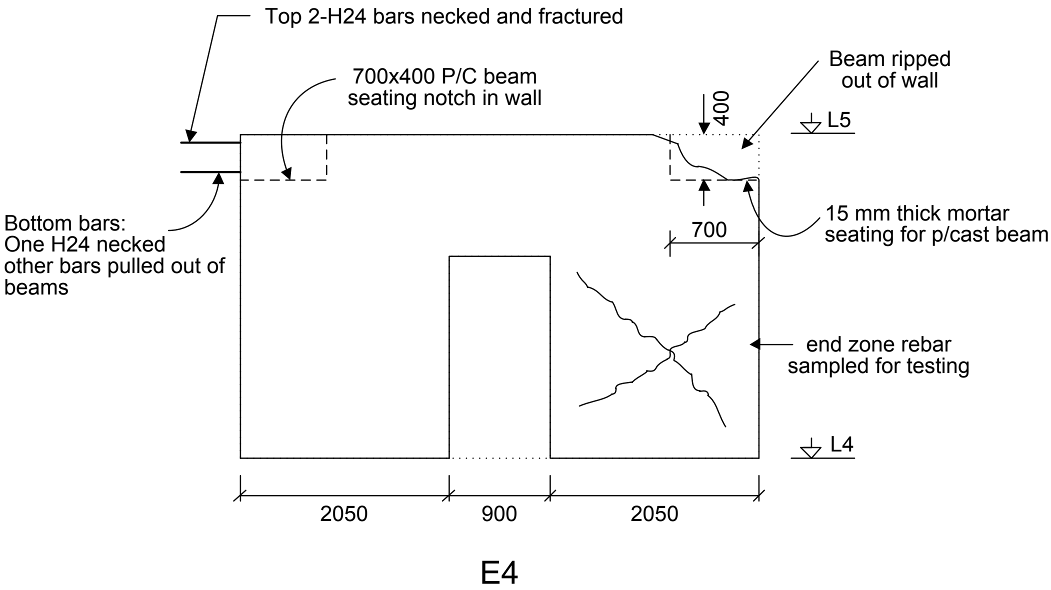

Figure 21 - South Wall Item E4 schematic of damage observed.

iv. Line 1 South Wall Level 5 to 6: Item E5

Figure 22 - Line 1 South Wall Level 5 to Level 6 (Item E5) (clockwise from top left) (a) Crumbly concrete at door edge of west pier able to be dislodged by boot; (b) Smooth and charred construction joint on top west surface looking east; (c) Charred construction joint above west pier. Door sill on left; (d); Top east corner with fractured top 3-H24 bars. Floor 664 mesh exposed.

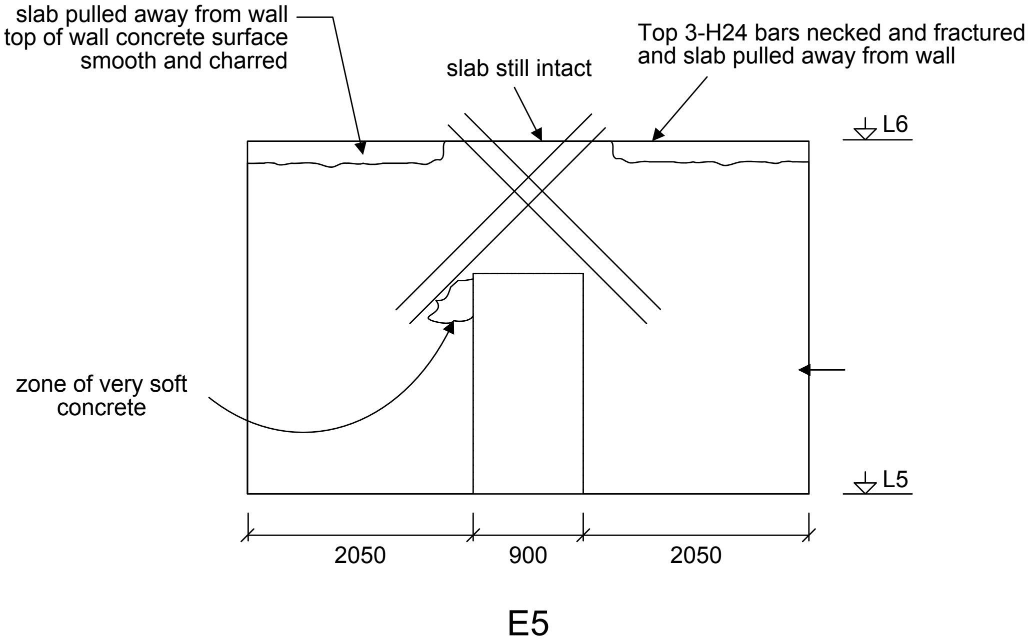

Figure 23 - South Wall Item E5 schematic of damage observed.

I. LINE 1 SOUTH WALL LEVEL 6 TO ROOF: ITEM E5A

Figure 24 - Line 1 South Wall Level 6 to Roof (Item E5A) (clockwise from top left) (a) Outer face; (b) Top surface at roof; (c) East pier with saw-cut from de-construction; (d) West pier at construction joint

J. NORTH CORE

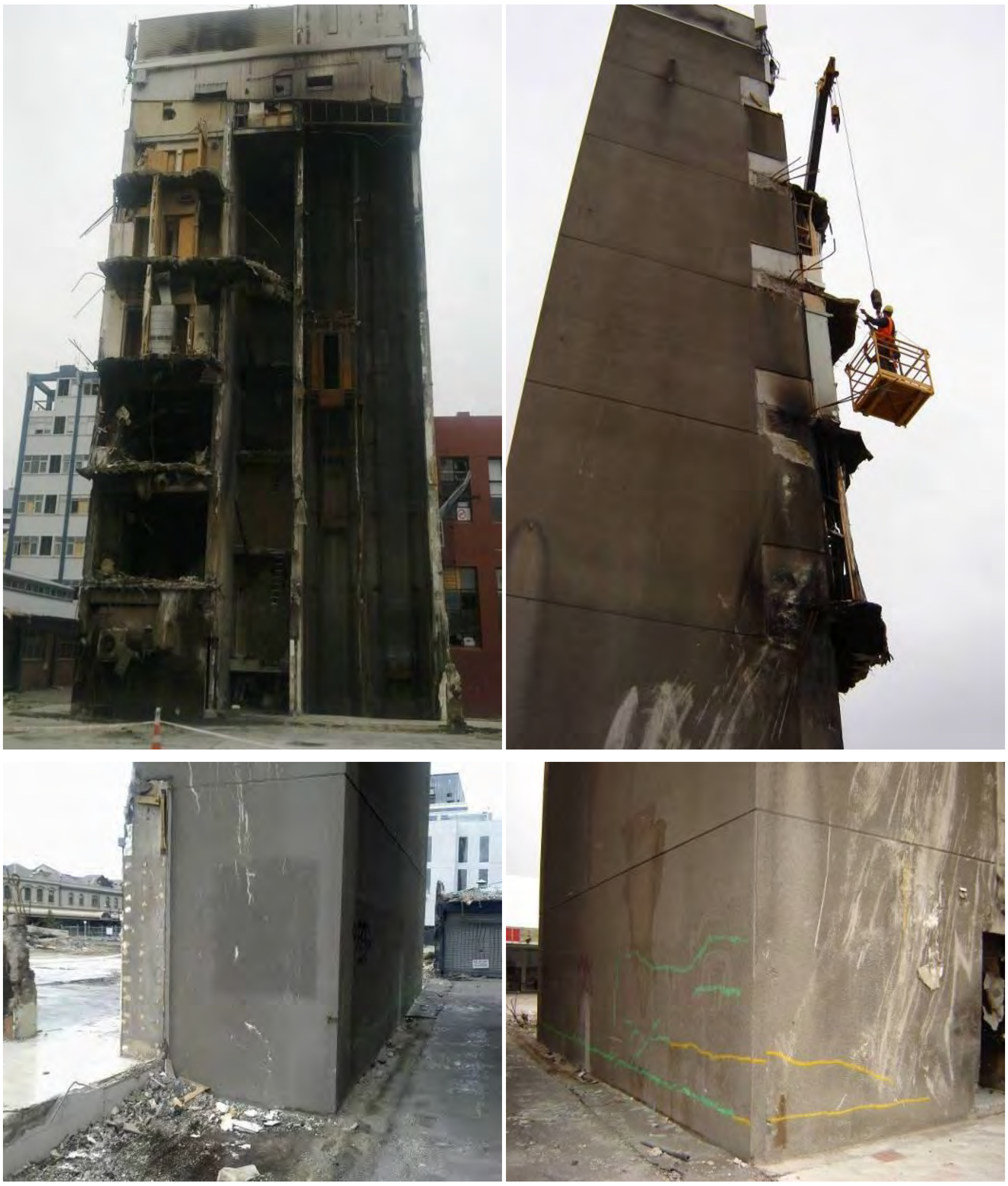

Figure 25 - Line 4 to 5 North Core (DENG Dwg S15 (Figure 65)) (clockwise from top left) (a) South face after site cleared with lift shaft for two cars on right, stair well in middle and amenity rooms on the left; (b) West face; (c) West and north face at Grid C/5 corner Level 1 to 2 with horizontal flexural cracks and construction joint identified by paint; (d) East face with column C18 remnant at far left

Figure 26 - Lift and Stair well wall cracking Level 1 to Level 2 (clockwise from top left) (a) Lift wall face of Line D wall with fine diagonal shear cracking in both directions; (b) Lift wall face of Line 5 wall with fine diagonal cracking in both directions; (c) Stairwell area with steel stair stringer where concrete cores were extracted; (d) Stair well face of Line D wall with fine diagonal cracking in both directions.

K. NORTH CORE: SOUTH SIDE LINE 4

The North Core walls, and remaining slabs and attachments were examined from a man-cage on 12 March, 2011 and from a platform on 5 April, 2011. Observations and comments are included in the captions to the photos.

i. Level 6 Slab Remnants

Figure 27 - Line 4 Core Wall Slab Remnant at Level 6 amenity area (clockwise from top left) (a) Slab edge on stairwell wall looking west with H12 saddle bar exposed and ends of mesh below it; (b) Vertical concrete fracture surface with reinforcing mesh fractured; (c) Slab looking west with cores cut in floor for amenities; (d) Fractured mesh angled downwards; (e) Fractured slab edge looking east. Torn metal decking aligned approximately with concrete fracture edge; mesh at varying height within slab; (f) Cores for amenities at fracture edge.

ii. Level 5 Slab Remnants

Figure 28 - Line 4 North Core Wall Slab Remnants Level 5 (clockwise from top left) (a) West end with H12 bar ends and mesh at bottom of slab above ribs. It was required on the drawings to be located near the top surface (DENG Dwg S16 (Figure 75)); (b) Looking east. Reinforcing angled down; (c) Slab edge on Grid C west end. Mesh angled down in bottom of slab on top of ribs; (d) Fire charred vertical fractured slab edges adjacent to stairwell. Mesh located low down in slab at top of slab ribs. Edge of mesh sheet with closely spaced parallel wires exposed. Fractured wires can be seen from the lapped mesh below; (e) Cracking in slab running from cored holes;(f) Connection of shell beam to wall with two fractured H24 top bars and two de-bonded top bars. No bottom bars from shell beam embedded in wall (DENG B23 Dwg S18 (Figure 67), Detail 5 S19 (Figure 71)), corresponding to the bent back bottom steel in shell beam Item E23 (Figure 8).

iii. Level 4 Slab Remnants

Figure 29 - Line 4 North Core Walls Level 4 slab remnants (top to bottom) (a) Connection of shell beam to wall with two fractured H24 top bars and two de-bonded top bars. No bottom bars from shell beam embedded in wall but imprints from bars evident (DENG B23 Dwg S18 (Figure 67), Detail 5 S19 (Figure 71)), corresponding to the bent back bottom steel in shell beam Item E23 (Figure 8); (b) Fractured vertical face of slab at stairwell wall, with fractured slab support beam (DENG B25 Dwg S18 (Figure 67)) top bar and charred fracture surface; (c) Torn profiled metal deck sheeting de-bonded from slab but still fixed in at pre-cast beam support.

iv. Level 3 Slab Remnants

Figure 30 - Line 4 North Core Walls Level 3 slab remnants (clockwise from top left) (a) Connection of shell beam to wall with two fractured H24 top bars and two de-bonded top bars. No bottom bars from shell beam embedded in wall but imprints from bars evident (DENG B23 Dwg S18 (Figure 67), Detail 5 S19 (Figure 71)), corresponding to the bent back bottom steel in shell beam Item E23 (Figure 8); (b) Ash on slab;. Cored holes at fractured edge; (c) Torn profiled metal deck sheeting de-bonded from slab but still fixed in at pre-cast beam support (DENG B24 Dwg S18 (Figure 67)); (d) Profiled metal deck and slab from below supported on beam B24.

v. Level 2 Slab Remnants

Figure 31 - Line 4 North Core Walls Level 2 slab remnants (clockwise from top left) (a) Slab edge broken back during de-construction adjacent to stairwell wall; (b) Broken back slab with H12 saddle bars exposed. Masonry wall with separation along top course; (c) Switch room under Level 2 slab; (d) Connection of shell beam to wall with two fractured H24 top bars and one de-bonded top bars. Bottom bars from shell beam have been embedded in wall as specified (DENG B23 Dwg S18 (Figure 67), Detail 5 S19 (Figure 71)). Some diagonal beam–wall joint zone shear cracking can be seen in the wall end..

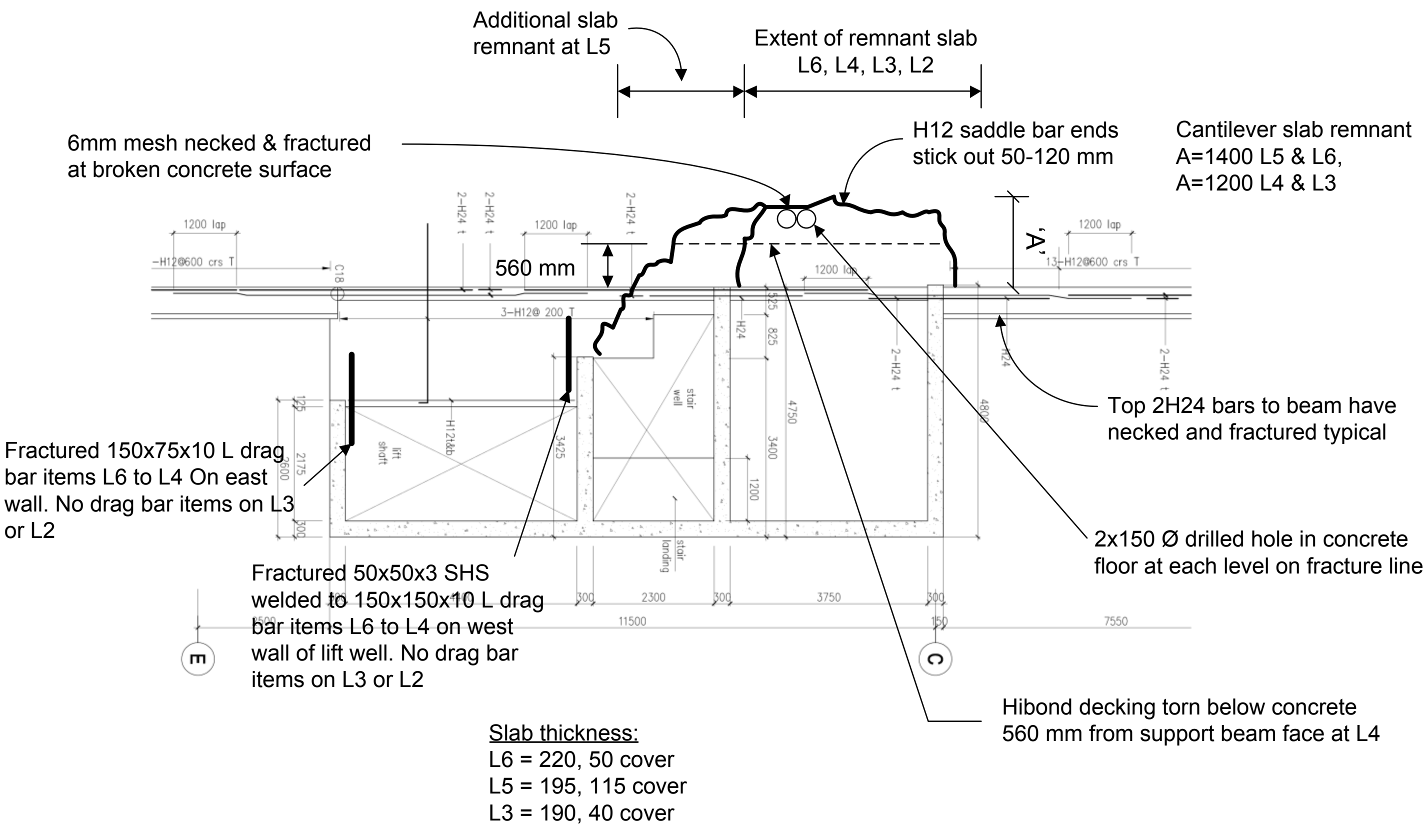

Figure 32 - Extent of remaining slab at time of site examination. Portions of the slab had been removed during deconstruction for safety reasons.

L. LIFT WELL WING WALLS DRAG BARS

The profiled metal deck floor slab at Level 4, 5 and 6 had additional Drag Bars connecting it to the north-south wing walls, ("the Drag Bars") on either side of the lift well, sometime after the original construction was completed. Threaded rods had been fixed into the slab with what appeared to be epoxy grout and bolted to the underside of the Drag Bars, as seen in Figure 36 to Figure 40.

Figure 33 - Drag Bar connections at Levels 6, 5 and 4 on lift well west wing wall on Grid D. No Drag Bar at Level 3 or 2 (Figure 32)

i. Level 2 Lift Well Wing Walls D and D/E

Figure 34 - Level 2 Lift Well Wing Walls Grid D and D/E (left to right) (a) 20 mm hole in end where a reinforcing bar has pulled out of wall. 200 mm thick construction joints in wall at slab level; (b) No reinforcing steel attachment into the east wing wall at Level 2

ii. Level 3 Lift Well Wing Walls D and D/E

Figure 35 - Level 3 Lift Well Wing Walls Grid D and D/E (anti-clockwise from top left) (a) Profiled metal deck decking side lapped into western Grid D wall, just hanging on; (b) H12 bar necked and fractured at centre of wing wall (indicated by chalk arrow); (c) Concrete cover appeared to have broken away as the slab pulled southwards; (d) Localised spalling of concrete. No reinforcing found connecting end of east wall with slab.

iii. Level 4 Lift Well Wing Walls D and D/E

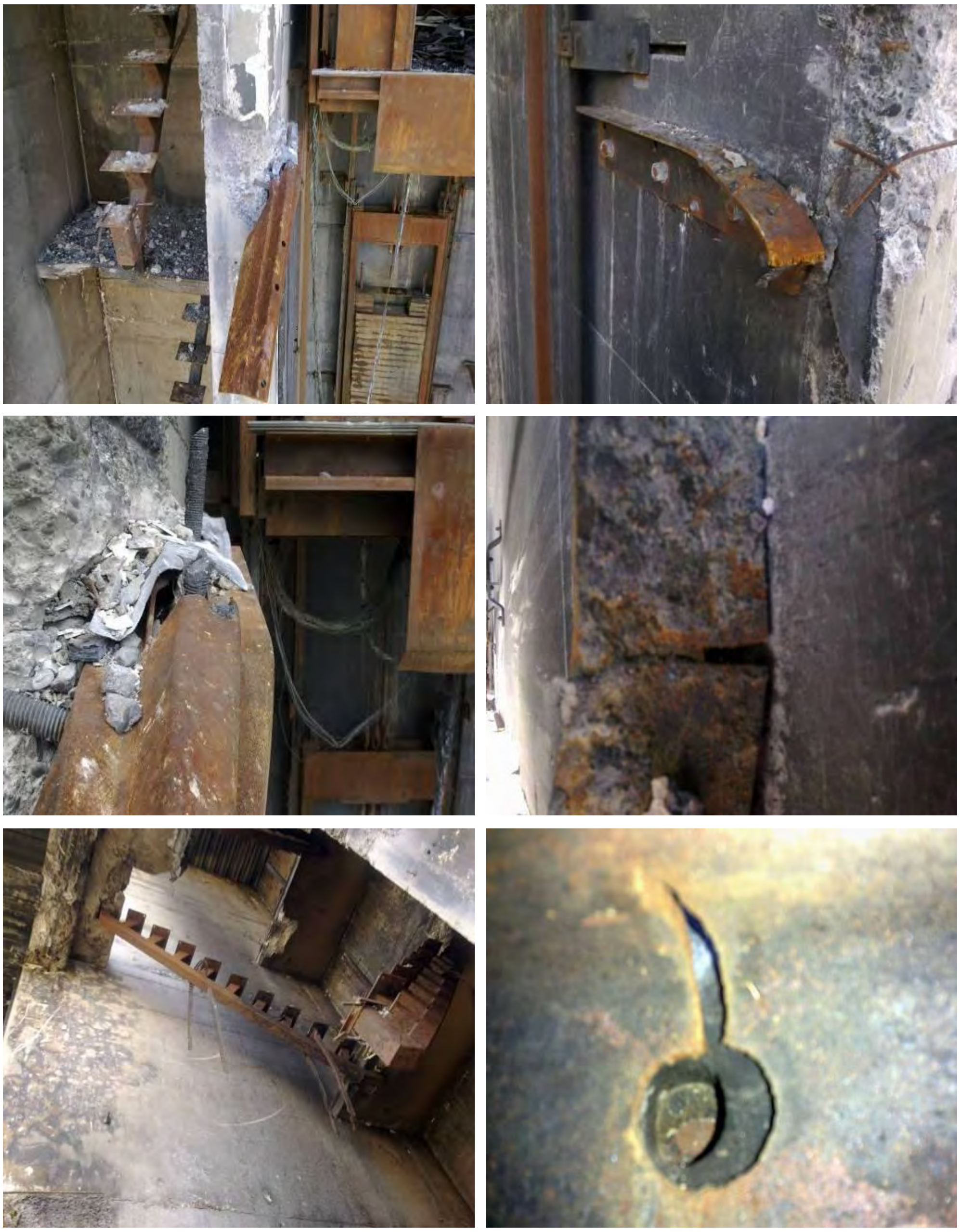

Figure 36 - Level 4 Lift Well Wing Walls Grid D and D/E (anti-clockwise from top left) (a) 150x150x10 L with 51 x 3.2 SHS; 3-M24 threaded rods fixed into wall and 6 –M20 threaded rods 350 mm long fixed into the slab at the profiled metal deck rib with epoxy grout around rod; (b) Three M20 threaded rods remained upright in the Grid D Drag Bar. The 51x3.2 SHS had fractured in bending and tension at the bolt hole adjacent to last bolt into wall; (c) Stair stringer running up to Level 5 fixed rigidly into landings with visible vertical bow (DENG Stair S8 Dwg S31 (Figure 76); (d) Initiation of fracture in the steel angle at the elongated hole without bolt into east wall; (e) Fracture in angle running from corner of angle out towards toe viewed from above; (f) Remnant of 150x75x10 L Drag Bar with 4-M24 threaded rods fixed into grid D/E wall. Angle has fractured in bending and tension two bolts in. End of Drag Bar had been gas cut during de-construction.

Figure 37 - M20 Drag Bar Bolts (Portions shown are from two different bolts) (left to right) (a) Grey portion with smooth diagonal end had epoxy residue from slab indicating it was cut this way prior to installation; (b) Necked and dimpled fracture surface at underside of bolt typical of direct tensile fracture in threaded rods.

Figure 38- M20 Property Class 5.8 threaded studs from Drag Bars after hardness testing by MTL (Figure 62)

iv. Level 5 Lift Well Wing Walls D and D/E

Figure 39 - Level 5 Lift Well Wing Walls Grid D and D/E (anti-clockwise from top) (a) 150x150x10 L with 51 x 3.2 SHS; 4-M24 bolts into wall and 6 –M20 threaded rods 350 mm long fixed through the slab at the profiled metal deck rib with epoxy grout around rod; Three M20 threaded rods remained vertical the Grid D Drag Bar. The 51x3.2 SHS had fractured in bending and tension at the bolt hole adjacent to last bolt into wall and twisted with the slab; (b) Epoxy grout around threaded rod in slab; (c) Holes for 3-M20 threaded rods in slab in twisted Drag Bar; d) 150x75x10 L Drag Bar with 5-M24 threaded rods fixed into wall D/E. End of Drag Bar had been gas cut during de-construction.

v. Level 6 Lift Well Wing Walls D and D/E

Figure 40 - Level 6 Lift Well Wing Walls Grid D and D/E (anti-clockwise from top left) (a) 150x150x10 L with 51 x 3.2 SHS into west wall on Grid D, similar to Level 5 and 4 Drag Bars; 6-M24 bolts into wall; The remaining 4 –M20 bolts 350 mm long had been fixed into the slab at the profiled metal deck rib with epoxy grout around bolt; These remained vertical and were measured with 110 mm stick-out above the item. The 51 x3.2 SHS has fractured in bending and tension at the bolt hole adjacent to last bolt into wall and the bar had twisted with the slab; (b) Side view of Drag Bar remnant showing deck uplift at end bolt; (c) Top surface of M20 threaded rod with typical smooth diagonal face and grout around rod through slab. (d) End rod with diagonal top surface and slab concrete remains; (e) 150x75x10 L Drag Bar with 7- M24 threaded rods fixed into east wall D/E. End of bar had been gas cut during de-construction.

M. LEVEL 7 LIFT WELL WING WALL D/E: COLUMN D/E-4 CONNECTION

Figure 41 - Lift Well Wing Wall D/E: Column D/E 4 Connection (DENG Dwg S14 (Figure 69); 3 x 20 to 24 mm diameter holes could be seen where reinforcing bars from the column had pulled out. The drawing shows that 4-H20 bars were required to be bent into the wall.

3. LEVELS AND POSITIONAL SURVEY

The remaining floor slab and lift core was surveyed by John Jones Steel Limited. A Transit Optical total-station laser levelling system was used. The total-station system gave levels and co-ordinates of the points shot with a stated accuracy of +/-5 mm.

The levels are relative to a temporary benchmark ("the TBM") set up near the lamp post on the kerb on the far side of Madras Street.

Shots were initially taken on 14 April, 2010 to the approximate centres of the demolished remains of the concrete columns (Figure 58).

Shots were then taken on 18 April, 2011 to pick out the edge of the slab overlay and up the sides of the west and east walls of the lift core (Figure 55, Figure 57).

Dumpy levels were subsequently taken 28 April, 2011 on the concrete adjacent to the columns. These were then identified as being either on the overlay, on the original nominal 125 mm slab or on the exposed foundation beams.

Photos of the column locations surveyed were also noted on the survey drawings.

The survey drawings, photos and analysis of levels is included in Appendix A.

A. FOUNDATION BEAM LEVELS

Analysis of the top of foundation levels, based on shots taken on foundation beams, showed an average level relative to the TBM of +3 mm, with a sample standard deviation of 16 mm from 6 shots.

The concrete construction standard NZS 3109:1980 allowed a level variation of +/-12 mm for top of foundations to receive in-situ construction.

B. SLAB LEVELS

The average RL of top of slab, which was cast directly on the top surface of the foundation beams was +120 mm with sample standard deviation of 12 mm from 12 shots. The nominal thickness of the slab specified was 125 mm cast to Finished Floor Level of 15.070 m (DENG Dwg S9 (Figure 64)). The average slab thickness calculated from the difference in the average RL of the slab and the foundation beams was 117 mm.

The variation in floor slab levels appeared to be consistent with the DENG specification section 2.8 (Figure 77) which required the floor slab to achieve a levelness tolerance of +/- 15 mm and flatness of +/-6 mm over 3m.

C. SLAB OVERLAY LEVELS

The average RL of top of the concrete overlay cast on top of the slab sometime after the original construction was +209 mm with sample standard deviation of 14 mm from 16 shots. It was not known what the specified nominal thickness of the overlay was. The average thickness estimated from the average RL of the slab and the overlay was 89 mm.

D. NORTH CORE NORTH FACE OUT–OF-PLUMB

The North Core wall on Line 5 was surveyed for verticality by sighting on the eastern and western corners of the north face of the wall. It was found that there was a northwards out-of-vertical measurement of 91 mm over 18.53 m between Level 1 and Level 7 at the northeast corner, and 68 mm over 18.53 m at the northwest corner (Figure 57).

This was greater than the plumbness limit of 25 mm for structures greater than 12m high in NZS 3109:1980.

E. CONCLUSIONS

The levels survey show that the foundation beams and original floor slab cast during initial construction had a variation in floor level after the Aftershock generally consistent with the specified tolerances.

The difference in average floor slab level and the foundation beam level resulted in an average derived floor slab thickness close to the specified slab thickness.

On this basis it was concluded that no slab or foundation settlement could be inferred to have occurred.

It is not known what caused the northwards out-of-plumb in the north face of the Line 5 wall of the North Core. However:

1. No evidence of settlement, uplift or rotation of the slab and foundation beams was found from the levelling survey.

2. No damage or cracking was found in the foundation beams running from Grid 3 to Grid 5 when the floor slab was removed for inspection as described in section 4.

3. No evidence of liquefaction was found around the foundations and adjacent to Line 5 when a pit was dug adjacent to the footing at Grid C and 5, as described in section 4.

4. NORTH CORE FOUNDATION INSPECTION

The slab was removed and a pit dug adjacent to the northwest corner of the North Core (Grid C/5) on 10 May 2011, to look for damage in the foundation beams around the lift core area and signs of liquefaction (Figure 59).

Another pit was subsequently dug adjacent to the northwest corner of the lift core after the walls had been substantially demolished on 13 May 2011. The pit was dug to check the side of the foundation for cracking. The remains of rotted timber boxing had to be removed from the concrete.

Nothing unusual was observed by the CERA engineer who undertook the inspections. No cracking damage was apparent in the foundation beams. No signs of liquefaction were found.

Notes and photos from the inspections are included in Appendix B.

5. REINFORCING STEEL PROPERTIES

A. SAMPLE LOCATIONS

Reinforcing steel was taken from structural remnants to identify typical material properties, and in the case of the H28 bars in the ends of the Line 1 South Wall, to identify if any yielding had occurred.

i. H16: Line 1 South Wall Level 1 Door Infill

Figure 42 - Line 1 South Wall Item E1 H16 bars from masonry door infill.

ii. H28: Line 1 South Wall Ends Level 1 Item E1

Figure 43 - H28 from east and west end of Line 1 South Wall Item E1 (clockwise from top left) (a) H28 about to be cut from east end (E1E); (b) Top of 1000 mm long E1E sample1300 mm from top of L2 slab; (c) Top of 1000 mm long E1W sample from west end 750 mm from top of L2 slab. Coupling beam depth was 1700 mm.

iii. H24: Line 1 South Wall Ends Level 3(E3) and 4 (E4)

Figure 44 - Locations of H24 reinforcing bar samples (left to right) (a) East end of Line 1 South Wall item E3, one cut from lower 1050 mm of wall; (b) Two lapping bars from lower E3 wall item taken from east end of wall item E4

B. 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 P5665 is included in Appendix C.

Tensile test results have been reported in accordance with the method of AS/NZS 4671:2001 (SNZ 2001). A summary of the tensile test properties is shown in Table 1.

Deformation measurements were also reported.

The tensile properties of the 16, 24, and 28 mm bars were very similar, whereas the 12 mm bars have greater yield and tensile strength properties.

The properties of the H28 bar E1E taken from wall E1 on the east end had elongation at maximum force (A_{gt}), 3.3% less than that of the H28 bar extracted higher up the wall on the west side E1W. It also had a measured yield stress of 464 MPa which is 17 MPa higher. This indicated that the E1E bar may have undergone a level of plastic work hardening. The E1W bar and the other 16 and 24 mm bars tested appear to have remained elastic due to the consistency of their maximum elongation values and yield stress.

The E1W bar had a yield stress R_e and elongation at maximum load A_{gt} very similar to the 16 mm, and 24 mm bars tested.

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

Size	Uniform Elongation A_{gt} (%)	Yield Stress Re: R_{eL} or $R_{0.2p}$ (MPa)	Ultimate Tensile Strength R_m (MPa)	Ratio R_m/R_e	Comments
12	16.0	518	677	1.31	Item E4
16	16.3	450	595	1.32	Item E1
24	17.2	446	607	1.36	Items E3 & E4
28	16.8	447	612	1.37	Item E1 specimen E1W only
16-28	**16.8**	**448**	**603**	**1.34**	**Average excluding specimen E1E**
28	13.5	464	627	1.35	Item E1 specimen E1E only
664 Mesh	4.2	615	665	1.08	Suspended floor slab

Table 1 - Summary of reinforcing steel tensile test results

C. CHEMICAL ANALYSIS

Reinforcing steel samples were sent to Pacific Steel Group laboratories in Otahuhu ("Pacific Steel") 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 2.

The Pacific Steel metallurgist advised that the results were consistent with them being from the same or similar production runs, and were within the variances expected from product testing.

The chemical analyses showed the bars tested to be conforming to Grade 380 reinforcing steel in accordance with NZS3402P:1973 Hot Rolled Steel bars for the Reinforcement of Concrete (SNZ 1973).

Sample	C	Mn	Si	S	P	Al	Ni	Cr	Mo	Cu	Sn	V	WCE
E1C H16	0.19	1.19	0.28	0.033	0.028	0.001	0.08	0.07	0.013	0.28	0.022	0..040	0.434
E4A H24	0.19	1.19	0.29	0.031	0.031	0.001	0.09	0.08	0.011	0.28	0.023	0.042	0.442
E4B H24	0.20	1.21	0.30	0.034	0.032	0.002	0.09	0.08	0.011	0.28	0.023	0.043	0.454
E1E H28	0.21	1.30	0.35	0.020	0.018	0.002	0.08	0.09	0.011	0.25	0.041	0.042	0.473
E1W H28	0.21	1.26	0.33	0.019	0.011	0.001	0.08	0.06	0.011	0.20	0.036	0.045	0.461

NB *-All figures are weight percentage values*

Table 2 - Chemical analyses of reinforcing bar samples by Pacific Steel

6. CONCRETE PROPERTIES

A. CONCRETE TESTING

Concrete cores were extracted at the CTV site from suspended slabs in two locations (Item E14 and E23); the Line 5 wall at Level 1of the North Core; the Line 1 South Wall between Level 4 and 5 (Item E4); the 400 mm diameter column between Level 6 and 7 (Item E25) (Figure 46); and the Level 1 400 mm square column from Line 4 – D/E (C18) stub (Figure 47).

Thirteen 400 mm diameter circular and twelve 400 x 300 mm rectangular concrete column remnants were extracted from the CTV debris located in the specially designated area at the Burwood Eco landfill (Figure 48).

The circular columns tested at the landfill are designated in this section of the report by a prefix "tC" (eg tC3 is 400 mm diameter circular column test item C3), and the rectangular columns are similarly designated by a prefix "tR".

The columns were selected from all over the CTV debris lot and were all that could be found remaining on the surface of the debris piles after walking systematically over the debris. It could be considered therefore that the column remnants were randomly selected.

Column Item E25 that had previously been cored at the CTV site was among those found and is identified as tC1 in the results table in Appendix C.

The column bases could be identified by the presence of terminating vertical lap bars.

Concrete compressive drilled core and rebound hammer testing was undertaken for slabs, beams and columns by Opus International Consultants Christchurch Laboratory ("Opus") (Jones 2011). Concrete compressive and chord modulus of elasticity was undertaken for shear wall cores at Opus Central Laboratories in Wellington (Wong 2011).

The sample means of the test results for a particular member were assessed against the known means of concrete properties with 28-day strengths conforming with NZS 3104:1983. A lower 0.1% acceptance limit was applied to identify upper bound conformity with a specific strength category. Where the sample size was sufficiently large an upper 0.1% rejection limit was also applied to identify non-conformity with a lower strength category.

Sample mean strengths were increased by 8% where testing orientation was transverse to casting direction, in accordance with Concrete Society recommendations (GBCS 1987).

Results are shown in Appendix C Table 6.

The concrete test results need to be interpreted in light of the fact that the samples were extracted from components that had been damaged in the collapse. Care was taken however to core and undertake rebound hammer tests in portions of concrete away from areas of obvious cracking and damage. All the cores were visually scanned before testing for signs of cracking and the requirements of the NZS 3112:1986.

i. Rebound Hammer Testing and Coring to ASTM C805

Schmidt or Rebound Hammer testing of the columns remnants was undertaken by OpusL on 30 May, 2011 (Jones 2011). Testing was in accordance with ASTM C805 (ASTM 2008) on the column remnants at locations near the top, middle and bottom ends of the specimens where possible.

Two cores were subsequently extracted and tested from each of five column test locations that had been found to have average hammer numbers approximately equal to the mean ,and 1 and 2 standard deviations either side of the mean (Jones 2011). This was to allow a relationship to be developed between the compressive test results for the cores and the hammer numbers in accordance with the requirements of ASTM C805 .

ii. Determination of Strengths from Hammer Numbers

The compressive strengths from Hammer Numbers were determined using the specific strength vs hammer number relationship developed by correlating the cored test results at 6 locations with the rebound hammer numbers in accordance with the ASTM C805 (ASTM 2008).

The rebound hammer manufacturer's concrete cylinder compressive strength curves were reviewed but found to be unreliable for this concrete. This is an issue identified by ASTM C805 with instrument manufacturer rebound hammer curves for concrete. This is because the strength to hammer number relationship varies with concrete mixes (cl. 5.2 ASTM C805). The charts however did provide a basis for assessing the relative effect of hammer orientation to the vertical on hammer numbers. At an angle +/- 45 o from vertical down ie 1030 to 0130 hr on a clock-face, the hammer number increases by 0.5 at HN=45 and by 0.8 at HN=35.

B. ALLOWANCE FOR STRENGTH-AGING EFFECT OF CONCRETE

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. It found that concrete batching practice typically sought to achieve 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).

The strength distribution assumed for ready –mixed New Zealand concrete 28-day strengths at the time of the CTV Building construction was NZS 3104:1983 Table 5 (SNZ 1983). This is plotted in Figure 51 with the actual test strength distribution.

It is widely understood in the construction industry that concrete properties can be adversely affected by concrete placement techniques, and these effects have not been accounted for in NZS 3104:1983 Table 5.

The statistical properties of the same concrete strength-aged were derived by application of a factor of 1.25 to the mean and standard deviation of the 28-day strength properties (Table 3). The 28-day strength properties strength-aged by 25% are plotted in Figure 52 with the actual test strength distribution.

Concrete Specified Grade Properties at 28 days and Strength-aged by 25%								
Variability of 28 day cylinder strength from Table 5 NZS3104:1983								
Specified 28 day Strength		**12.5**	**15.0**	**17.5**	**20.0**	**25.0**	**30.0**	**35.0**
	Lower 5%	12.5	15.0	18.7	20.7	25.8	31.5	36.5
	Lower 0.1%	8.4	10.3	13.2	15	19.2	24.2	28.9
	Upper 99.9%	26.1	30.6	36.8	40	47.8	55.8	62.1
	Target Mean	17.3	20.5	25.0	27.5	33.5	40.0	45.5
	cov	0.168	0.163	0.155	0.150	0.140	0.130	0.120
	standard deviation	2.90	3.32	3.87	4.13	4.70	5.20	5.46
Strength-aged by 25%	**Lower 5%**	**15.6**	**18.7**	**23.3**	**25.9**	**32.2**	**39.3**	**45.7**
	Lower 0.1%	10.6	12.9	16.5	18.7	24	30.2	36.1
	Upper 95%	27.5	32.4	39.2	42.8	51.5	60.7	68.1
	Upper 99.9%	32.6	38.2	46.0	50.1	59.7	69.8	77.6
	Mean	21.6	25.6	31.3	34.4	41.9	50.0	56.9
	cov	0.168	0.163	0.155	0.150	0.140	0.130	0.120
	standard deviation	3.62	4.16	4.84	5.16	5.88	6.50	6.82

Table 3 28-day concrete strength properties as per NZS 3104:1983 and also aged 25%

C. SUSPENDED SLAB COMPRESSIVE TEST PROPERTIES

Average compressive strength from three cores each for the slabs attached to Items E14 and E23 was 27.3 MPa and 22.0 MPa respectively. Tests were all taken parallel with the concrete casting direction.

The specified concrete for the slabs was 'high grade' in accordance with NZS3109:1980 (SNZ 1987), with a compressive strength was f'c = 25 MPa at 28 days.

The mean of 24.6 MPa (n=2) of the suspended slab concrete is greater than the lower 0.1% confidence limit of 23.4 MPa for concrete with specified 28-day strength of 25 MPa. It is also greater than the lower 0.1% confidence limit of 23.6 MPa for concrete with 28-day strength of 20 MPa strength-aged by 25%.

In conclusion, at the time of testing the concrete in the suspended slab remnants had mean strength not greater than that of concrete with 28-day strength of 25 MPa, or of 28-day strength of 20 MPa Aged by 25%.

This indicates that at the time of the collapse the concrete in the slab may have had strength consistent with that specified.

It also may indicate that the concrete used to construct the slab may not have achieved the specified 28-day strength of 25 MPa at the time of construction.

However it is recognised that the portions of slab that were tested had significant damage.

D. PRECAST INTERNAL BEAM

The single test core had a strength of f'c =25.0 MPa. Adjusted 8% for testing orientation transverse to casting direction this was 27.0 MPa.

This is greater than the lower bound 0.1% acceptance limit of 24.2 MPa for concrete with 28-day strength of 30 MPa. It is also greater than the 0.1% acceptance limit of 24.0 MPa for concrete with 28-day strength of 25 MPa Aged by 25%.

This seems consistent with the precast beams having achieved the specified 28-day strength of 25 MPa at the time of construction.

E. SOUTH WALL AND NORTH CORE CONCRETE TEST PROPERTIES

i. South Wall and North Core Compressive Test Properties

Average compressive strength from the four cores for the South Wall on Line 1 (Item E4) was 32.0 MPa and for three cores on Line 5 of the North Core was 35.5 MPa.

The specified concrete for the walls was 'high grade' in accordance with NZS3109:1980, with a compressive strength was f'c = 25 MPa at 28 days.

The mean of the shear wall concrete was 33.8 MPa for the two locations. (n=2).

When adjusted 8% for testing orientation transverse to casting direction this became 36.5 MPa. This is greater than the lower 0.1% confidence limit of 33.8 MPa for concrete with the specified strength of 35 MPa. It is also greater than the lower 0.1% confidence limit of 36.0 MPa for concrete with the specified 28-day strength of 30 MPa strength-aged by 25%.

In conclusion, at the time of the testing the concrete from the shear walls had mean strength not greater than that of concrete with 28-day strength of 35 MPa, nor 28-day strength of 30 MPa Aged by 25% or less.

This seems consistent with the South Wall and North Core having achieved the specified 28-day strength of 25 MPa at the time of construction.

ii. Shear Wall Chord Modulus of Elasticity Test Properties

The average shear wall chord modulus of elasticity was determined in accordance with AS1012.17-1997.

For six cores extracted from the shear walls on Line 1 (Item E4) and Line 5 Lift Core Walls the average was 27,600 MPa, with a minimum of 24,000 and maximum of 29,000 MPa.

iii. Shear Wall Secant Modulus of Elasticity

The average compressive strength from the seven cores for the shear walls on Line 1 (Item E4) and Line 5 Lift Core Walls was 33.5 MPa.

Using this value in the secant modulus equation of clause 5.2.3 NZS 3101:2006 the mean secant modulus of elasticity is calculated to be 26,100 MPa.

F. COMPRESSIVE TEST PROPERTIES OF CONCRETE COLUMN REMNANTS

Column test properties and observations of their failure condition are shown in Appendix C Table 6.

All column concrete testing was undertaken by Opus Laboratories Christchurch, in accordance with NZS 3112:1986 (SNZ 1986).

All cores were cut and testing undertaken transverse to the direction of concrete casting. Therefore an 8% adjustment to test strength values was made in accordance with the recommendations of Concrete Society Technical Report No.11 (GBCS 1987).

Cores were extracted to avoid areas of obvious cracking in the damaged members. All tests were visually scanned for signs of visible cracking prior to testing.

Only column C18 was obviously affected by fire amongst the columns tested.

i. Description of Circular Column Remnants

Circular column remnants tC1, tC4 and tC5 were full height column remnants with hinging at the base if a Level 6 to roof column, or at the base and top otherwise (Table 6).

Column remnant tC1 (Item E25) was a full height column from Level 6 to Roof with hinging at its base and still connected by reinforcing steel to column remnant tC8 which had hinging at its base and failure also at mid-height, with all the concrete in between gone.

Circular column remnants test Items tC7, tC8, tC9, tC11, tC12 and tC13 had flexural hinging zones at their bases around the lapping bars and hinging failure similar to that seen in Item 33 which was a perimeter column (Figure 53) and Figure 10). In that case spear head style fracture commenced approximately 1350 mm above the base and terminated at around 1600 mm. This may have coincided with the end of the column lap bars, which were specified to be 1200mm long (DENG Dwg S14 Figure 69).

Column remnant tC6 also had column mid-height failure though the top of it was connected into the roof. Column remnant tC3 had similar failure at one end like the other but at the lower end the bars had been cut off during de-construction.

The pre-cast spandrel panels may have had some influence on the mid-height failures by inducing short column effects (Figure 5 and Figure 72). The 400 mm diameter columns that suffered the mid-height failures may therefore have been from Grids 1, 4 and F like Item 33.

Circular column remnants tC2 and tC10 were able to be identified as Level 1 to 2 columns at the Grid 4 F entry (DENG Dwg S14 C23, and C21 or C22). These were the only columns specified as having 6 D12 vertical bars and tC10 had a downpipe cast into it (Figure 54).

The specified concrete 28-day strength for these two Level 1 columns was 35 MPa at 28 days according to the Specification. However the mean strength was 24.9 MPa (26.9 MPa when adjusted for test orientation) based on the Rebound Hammer tests, so appeared to have not complied with the requirements for 35 MPa concrete. Photos, taken by a member of the public during construction, show that these columns were not cast at the same time as the other Level 1 columns (Figure 45).

Figure 45 - CTV Building under construction (left to right) (a) May 1987 with floors cast up to Level 4; (b) October 1987 with roof on and pre-cast spandrel panels attached; Columns C21 to C23 had not been built at that time in the northeast corner closest to camera.

ii. Description of Rectangular Columns Line A

Columns tR1, tR2, tR3, tR4and tR4', and tR5 were full height and showed hinging at the base and tops typically where the beam-column joint had failed and the beam had pulled away.

Columns tR4 and tR4' were lower and upper columns running between Level 5 to Level 6 and the Roof respectively still connected by reinforcing steel.

Columns tR7 and tR6, and tR8 and tR9 were also lower and upper columns respectively running between two unknown levels still connected by reinforcing steel through failed beam-column joint zones.

Columns tR6, tR7, tR8, tR10 and tR10' had beam-column joint failures at the base and mid-height hinging.

The bottom of the Column tR9 had a smooth flat surface as would have been obtained from an un-roughened construction joint at floor level.

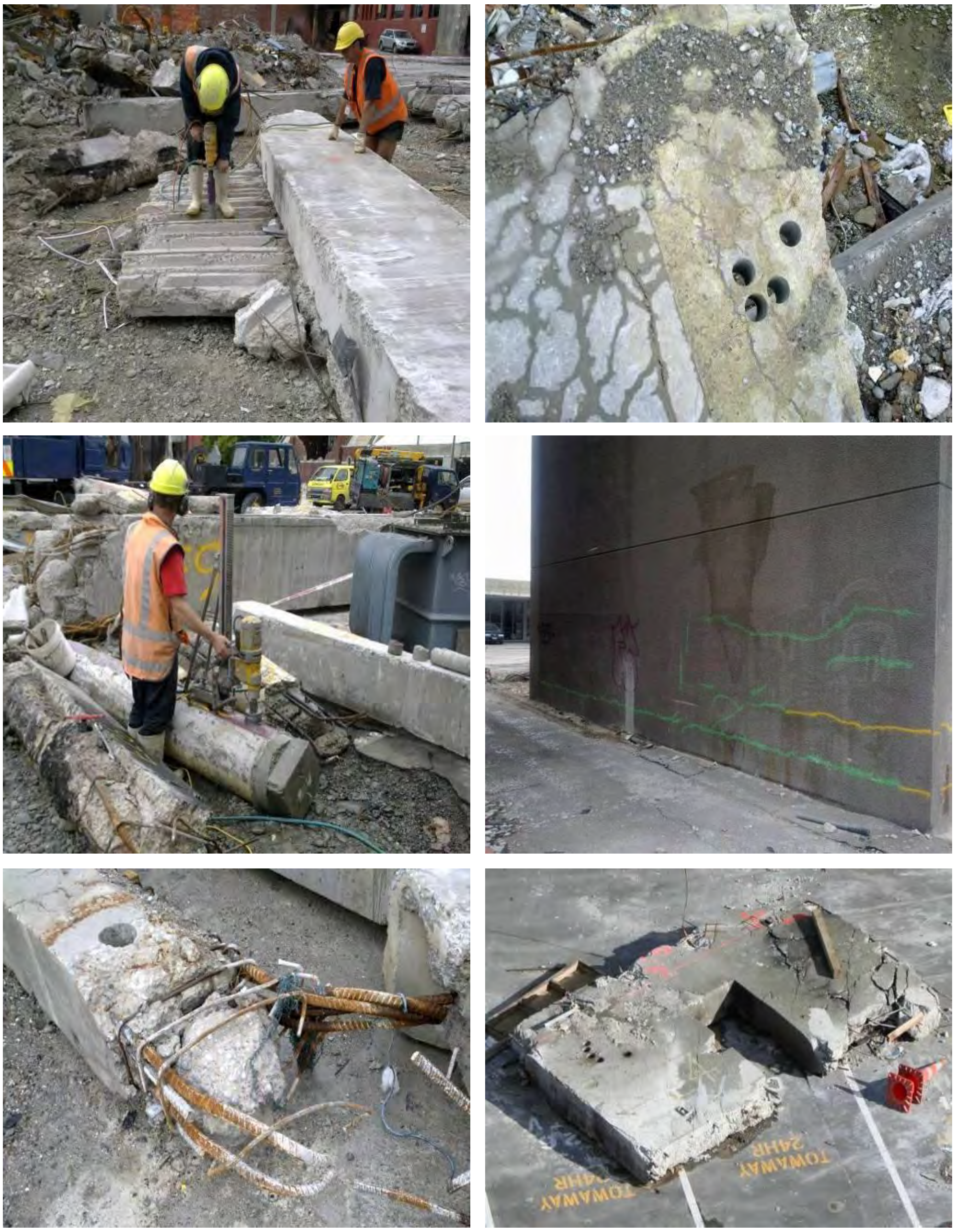

Figure 46 – Locations of concrete cores taken for testing (clock-wise from top left) (a) Slab attached to Line 1 or 4 perimeter shell beam (Item E23) (b) Slab attached to Line 4 perimeter shell beam (Item E14); (c) Line 5 shear wall through stair well at Level 1; (d) Line 1 South wall Level 4 to 5 (Item E4); (e) Pre-cast log beam from Line 2 or 3; (f) Level 6 400 mm diameter column (Item E25)

Figure 47 – Locations of concrete cores taken on site for testing on column C18 on Line 4, adjacent to wall D/E

Figure 48 - CTV Columns remnants extracted from the Burwood Landfill CTV debris (at right) for Schmidt Hammer testing and coring. Full height and partial height remnants can be seen.

Figure 49 -400 mm diameter columns full height (left to right) (a) Test Item tC1 Level 6 to roof column with base hinging failure, still connected by reinforcing to tC8 below; (b) tC4 hinging top and bottom; (c) tC5 Level 6 to roof column with hinging at base.

iii. Level 1 400mm Square Column C18 at Line 4 D/E

This column initially presented difficulties in gaining suitable uncracked cores, possibly due to them being taken horizontally and too close to the fractured zone (Figure 47). However as it was a Level 1 column the remnant was cut out and taken back to the testing laboratory and complying cores were able to be extracted for testing in accordance with NZS 3112:1986 (SNZ 1986).

This column had been affected by fire so efforts were made to take cores away from the heat affected cover concrete and also trim off cover concrete before testing.

Average compressive strength from the six cores tested (n=6) for the Level 1 square column C18 (DENG Dwg S9 (Figure 64) and S14 (Figure 68 and Figure 69)) was

16.0 MPa, with a minimum of 11.0 and maximum of 25.1 MPa. The mean test strength adjusted 8% for testing orientation was 17.3 MPa.

The specified concrete for the columns founded at Level 1 was 'high grade' in accordance with NZS3109:1980, with 28-day strength of 35 MPa (Figure 77).

The lower 0.1% sample mean acceptance criteria of 16.3 MPa indicates that the column remnant had mean concrete strength not greater than that of concrete with 28-day strength of 15 MPa.

Opus pointed out areas of possible silt contamination in the samples of column C18.

In conclusion, at the time of the tests, subject to there being no detrimental effects on the concrete test samples from heat from the post-collapse fire, the Level 1 400 mm square column concrete would not have complied with the requirements of concrete with the specified 28-day strength of 35 MPa nor 15 MPa. However it is recognised that this remnant had been affected by heat and collapse damage.

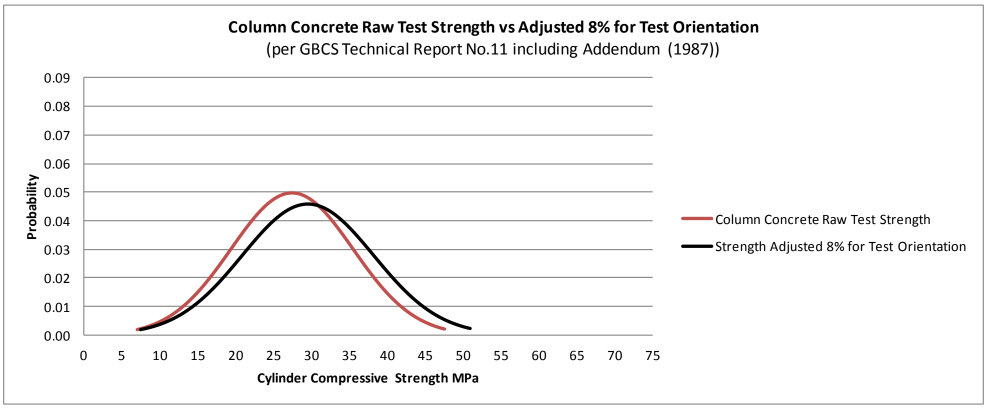

Figure 50 Column concrete test strengths compared to strengths adjusted 8% for test orientation being transverse to direction of concrete casting. This adjustment in test strength was recommended by the Concrete Society Technical Report 11 "Concrete Core Testing for Strength".

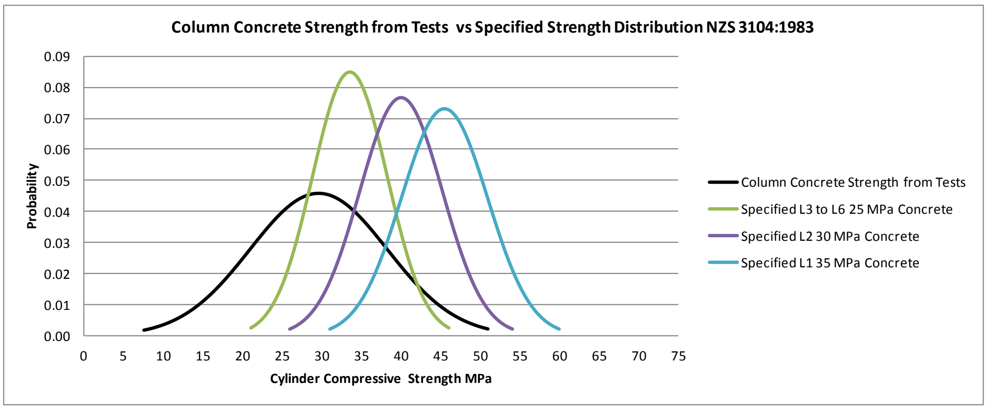

Figure 51 Column strengths from tests adjusted 8% for test orientation vs 28-day concrete strength distribution according to NZS3104:1983

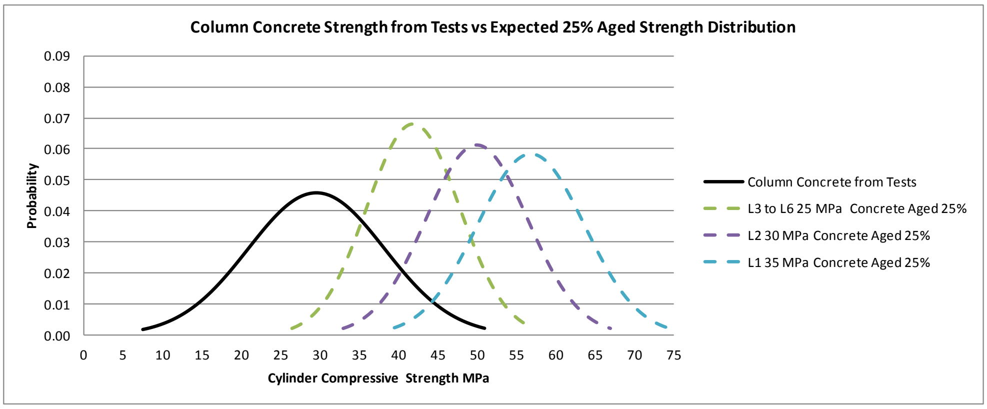

Figure 52 Column test strengths adjusted 8% for test orientation vs 28-day concrete strength distribution according to NZS 3014:1983 strength-aged by 25%

iv. Level 6 400mm Diameter Column (E25)

Average compressive strength from the three cores for the Level 6 column (Item E25) was 23.3 MPa. Adjusted 8% for test orientation this was 25.2 MPa.

The specified concrete for the columns at and above Level 3 was 'high grade' in accordance with NZS3109:1980, with 28-day strength of 25 MPa.

The sample mean of 25.2 MPa for 3 tests of the 400 mm diameter column is greater than the 0.1% lower confidence limit of 25.2 MPa for concrete with 28-day strength of 25 MPa. It is also greater than that of 22.8 MPa for 17.5 MPa aged by 25%.

This indicates that at the time of testing the column remnant had mean concrete strength not greater than that consistent with concrete with specified 28-day strength of 25 MPa or 17.5 MPa aged by 25%.

v. Level 1 Column Properties

Columns able to be specifically identified as being from Level 1 had a sample mean of 21.9 MPa (n=3). Adjusted 8% for testing orientation this was 23.7 MPa.

This was greater than the lower 0.1% confidence limit of 20.3 MPa for concrete with 28-day strength of 20 MPa, and greater than 22.8 MPa for 17.5 MPa aged by 25%.

The specified 28-day strength was 35 MPa.

vi. Level 1 to 6 Column Concrete Properties

Concrete cores and calibrated Rebound Hammer tests on 26 columns out of a possible 123 found an average strength of per column of 27.4 MPa. Adjusted 8% for testing orientation effects this was 29.6 MPa. The sample statistics are shown in Table 4. These have been calculated using the n-1 method to adjust for sample size.

The adjusted sample mean of 29.6 is greater than the 0.1% lower confidence limit of 25.0 MPa for concrete with 28-day strength of 20 MPa. It is also greater than the upper 99.9 % confidence limit for concrete with 28-day strength of 17.5 MPa of 27.3

MPa and less than the lower 0.1% confidence limit for concrete with 28 strength of of 25 MPa of 30.7 MPa.

This indicates that at the time of the testing the column remnants from Levels 1 to 6 had mean concrete strength consistent with that of concrete with specified 28-day strength of 20 MPa (Figure 51 and Figure 52). This is less than the minimum specified concrete 28-day strength of 35 MPa for columns at Level 1; 30 MPa for columns at Level 2; and 25 MPa for columns from Level 3 to Level 6.

The adjusted sample mean of 29.6 is greater than the 0.1% lower confidence limit of 28.4 MPa for concrete with 28-day strength of 17.5 MPa. It is also greater than the upper 99.9 % confidence limit for concrete with 28-day strength of 15.0 MPa of 28.0 MPa and less than the lower 0.1% confidence limit for concrete with 28 strength of of 20 MPa of 31.3 MPa.

This indicates that the concrete in the columns in Levels 1 to 6 may not have achieved strength consistent with that of concrete with specified 28-day strength of 25 MPa, at the time of construction. The minimum specified concrete 28-day strength was 35 MPa for columns at Level 1; 30 MPa for columns at Level 2; and 25 MPa for columns from Level 3 to Level 6.

However it is recognised that the tests were made on members that had suffered distress during the collapse.

	As-Tested	Adjusted 8% for Orientation
Sample Size (n)	26	26
Minimum (MPa)	16.0	17.3
Maximum (MPa)	46.6	50.3
Lower 5% (MPa)	14.2	15.3
Mean (MPa)	27.4	29.6
Upper 95% (MPa)	40.6	43.8
Coefficient of Variation (cov)	0.293	0.293
Standard Deviation (MPa)	8.04	8.68

Table 4 Column concrete test properties statistics

vii. Level 5 to 6 Column Properties

Columns able to be specifically identified as being from Level 5 and 6 had a sample mean of 25.1 MPa (n=8). Adjusted 8% for testing orientation this was 27.1 MPa.

This was greater than the lower 0.1% confidence limit of 23.1 MPa for concrete with 28-day strength of 20 MPa but less than that specified of concrete with 28-day strength of 25 MPa.

This indicates that at the time of testing the column remnants from Level 5 and 6 had mean concrete strength consistent with that of concrete with specified 28-day strength of 20 MPa.

This was greater than the lower 0.1% confidence limit of 26.1 MPa for concrete with 28-day strength of 17.5 MPa Aged by 25% and less than the lower 0.1% confidence limit of 28.8 MPa for concrete with 28-day strength of 20 MPa Aged by 25%.

This indicates that some of the concrete from columns in Levels 5 and 6 may not have achieved the specified 28-day strength at the time of construction. The minimum specified concrete 28-day strength was 25 MPa for columns from Level 3 to Level 6.

G. DISCUSSION OF CONCRETE COLUMN PROPERTIES

The DENG Specification required the concrete 28-day strength to be 35 MPa at Level 1, 30 MPa at Level 2 and 25 MPa at Level 3 and above.

The core and rebound hammer tests, were based on the testing of 26 column remnants selected at random from the debris. This is a significant sample size statistically and allows confidence to be taken in the results.

There was a marked difference in the tested core strengths of columns tR7 (40.9 MPa) and the tR6 (25.5 MPa). Column tR7 was the lower column of two, still attached by reinforcing steel to column tR6. It is therefore possible that column tR7 was cast using concrete with different specified 28-day strength than tR6. This may have been a column from Level 2 which had specified 28-day strength of 30 MPa or it may just be consistent with statistical variation in the concrete mix design.

However the specifically identifiable Level 5 and 6 (n=8) and Level 1 (n=3) and the unknown level (n=15) columns were found to have similar statistical characteristics as the total sample (n=26). The selection of the column remnants for testing was random and based on what was visible and retrievable from the debris field at the Burwood Landfill.

Figure 53 -400 mm Diameter Columns at Burwood showing similar base and/or mid-height failures (Left to right, top row down) (a) Test item tC6 Level 6 column head with mid-height failure; (b) tC7 with base and mid-height failures; (c) tC8 with base flexural (near end) and mid-height spearhead failures; (d) tC9 Level 6 column head with mid-height failure; (e) tC11 base (near end)and mid-height failure; (f) tC12 base and mid-height failure; (g) tC13 base and mid-height flexural failure(near end); (h) tC3 similar to tC13 but lower bars have been cut during de-construction at start of spalling.

Figure 54 - Level 1 Entry 400 mm Diameter Columns (left to right) (a) Test item tC2, 6-D12 vertical bars fractured at base (DENG Dwg S14 C23); (b) Item tC10 with down pipe cast in (DENG Dwg S14 C21 or C22)

Figure 55 -400 x 300 mm Rectangular Columns (Left to right, top down) (a) tR1 beam-column joint failure at base, mid-height failure; (b) Level 6 to Roof base or beam-column joint failure; (c) tR3 failure base and top; (d) tR4 Level 6 to Roof with beam-column joint failure, still connected by rebar to (e) tR4' below which also (f) indicates beam-column joint failure at tR4' base (near camera); (g) tR8 with damage from mid-height still connected to tR9 above (h) with beam-column joint failure with (i) underside of tR9 smooth.

Figure 56 -400 x 300 mm Rectangular Columns (from left to right and top down) (a) tR6 base at far end connected by reinforcing to (b) tR7 below, with beam-column joint failure; (c) tR10 remnant; (d) tR10' remnant

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7. CONCLUSIONS

In the opinion of the author the site examination and materials testing have resulted in the following conclusions:

1. The concrete test results need to be interpreted recognising that the samples were extracted from components that had been damaged in the collapse. Care was taken however to avoid coring in or undertaking rebound hammer tests on portions of concrete with obvious cracks. Cored samples were visually scanned before testing for signs of cracking and conformity with the requirements of the testing standard NZS 3112:1986.

2. The minimum specified concrete 28-day strengths were 35 MPa for columns at Level 1; 30 MPa for columns at Level 2; and 25 MPa for columns from Level 3 to Level 6.

3. Based on the testing undertaken, it appears that at the time of the testing the column remnants from Levels 1 to 6 had mean concrete strength consistent with that of concrete with specified 28-day strength of 20 MPa.

4. Based on the testing undertaken it appears that at the time of testing the concrete in the South Wall and North Core may have met the minimum 28-day strength specified of 25 MPa.

5. Based on the testing undertaken it appears that at the time of the testing the concrete in the suspended slab may have met the minimum 28-day strength specified of 25 MPa.

6. A portion of reinforcing steel removed from the Line 1 South Wall near ground level appeared to have "work hardened" during the Aftershock and prior to the collapse of the building.

7. No evidence of settlement of the foundations and slab was able to be inferred from the site levels survey which found levels consistent with construction practice at the time of construction.

8. The north face of the Line 5 wall of the North Core was found to be out of plumb by an amount greater than the construction tolerances allowed in NZS 3109:1980.

9. Construction joints and interfaces between pre-cast components and other concrete elements were found to be typically smooth rather than roughened as is normally required to improve interface interlock.

10. Reinforcing steel from several pre-cast shell beams was not developed into the Line 4 core wall as specified.

11. Connection of the slabs by reinforcing steel into the Line D and D/E walls of the North Core was non-existent in some cases at Level 2, 3 and 4.

12. The connection of the C18 column (located at Line 4-D/E) into the lift core wall at Level 7 was less than specified and the bars had de-bonded.

13. A number of circular columns examined showed mid-height hinging failures as well as hinging at the base or head. This was also seen in a column remnant clearly identified as being a perimeter column that had been located between precast spandrel panels. Other circular columns were found full height with hinging damage at the head and base.

14. Rectangular columns which had all been located on Line A in the structure, typically exhibited beam-column joint failure as well as other damage.

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APPENDIX A: LEVELS AND POSITIONAL SURVEY RESULTS

The levels taken on the foundation beams, top of slab and top of overlay which all had nominally the same top of concrete level have been analysed in Table 5.

The JJ Steel drawings have been annotated to identify whether the levels were taken on the foundation beams, slab or slab overlay (Figure 57 and Figure 58).

JJS Dwg	Local Coordinates from 4/F		Adjacent Grid or Feature		Location	Levels	Average	SD	Number
	West	South	Grid	Grid					
2	17027	21033	C.5	STEP	F	30			
1	20071	22537	C	1	F	5			
1	26209	22537	A.5	1	F	0			
1	30066	22467	A	1	F	0			
1	30067	7474	A	3	F	5			
1	30121	14983	A	2	F	-20	3	16	6
1	0	22507	F	1	O	182			
1	13	15005	F	2	O	197			
2	77	12850	F	2.5	O	201			
2	449	22744	F	1	O	195			
2	845	12868	F	EDGE OF OVERLAY	O	200			
1	4481	7447	E	3	O	210			
1	4507	15206	E	2	O	215			
2	6031	13667	E	EDGE OF OVERLAY	O	220			
1	6956	0	D.5	4	O	195			
1	11495	7501	D	3	O	220			
1	11507	14980	D	2	O	220			
2	11957	14251	D	2	O	229			
2	17264	18416	C.5	EDGE OF OVERLAY	O	220			
2	17812	12373	C.5	EDGE OF OVERLAY	O	200			
2	17894	6841	C.5	EDGE OF OVERLAY	O	225			
1	18472	7482	C.5	3	O	220	209	14	16
1	50	7479	F	3	S	100			
1	7498	22465	D.5	1	S	130			
1	12532	22465	D	1	S	115			
2	17027	21033	C.5	EDGE OF OVERLAY	S	100			
2	17264	18416	C.5	EDGE OF OVERLAY	S	110			
2	17812	12373	C.5	EDGE OF OVERLAY	S	122			
2	17894	6841	C.5	EDGE OF OVERLAY	S	122			
1	18472	7482	C	3	S	115			
1	18558	14992	C	2	S	130			
2	21794	20006	B.5	1.5	S	122			
2	22029	774	B.5	4	S	110			
1	25511	15001	B	2	S	125			
1	25515	7460	B	3	S	135			
1	26201	11	B	4	S	125			
1	30109	19	A	4	S	145	120	12	15

Table 5 - Relative levels of top of foundation (F), top of slab (S) and top of overlay (O)

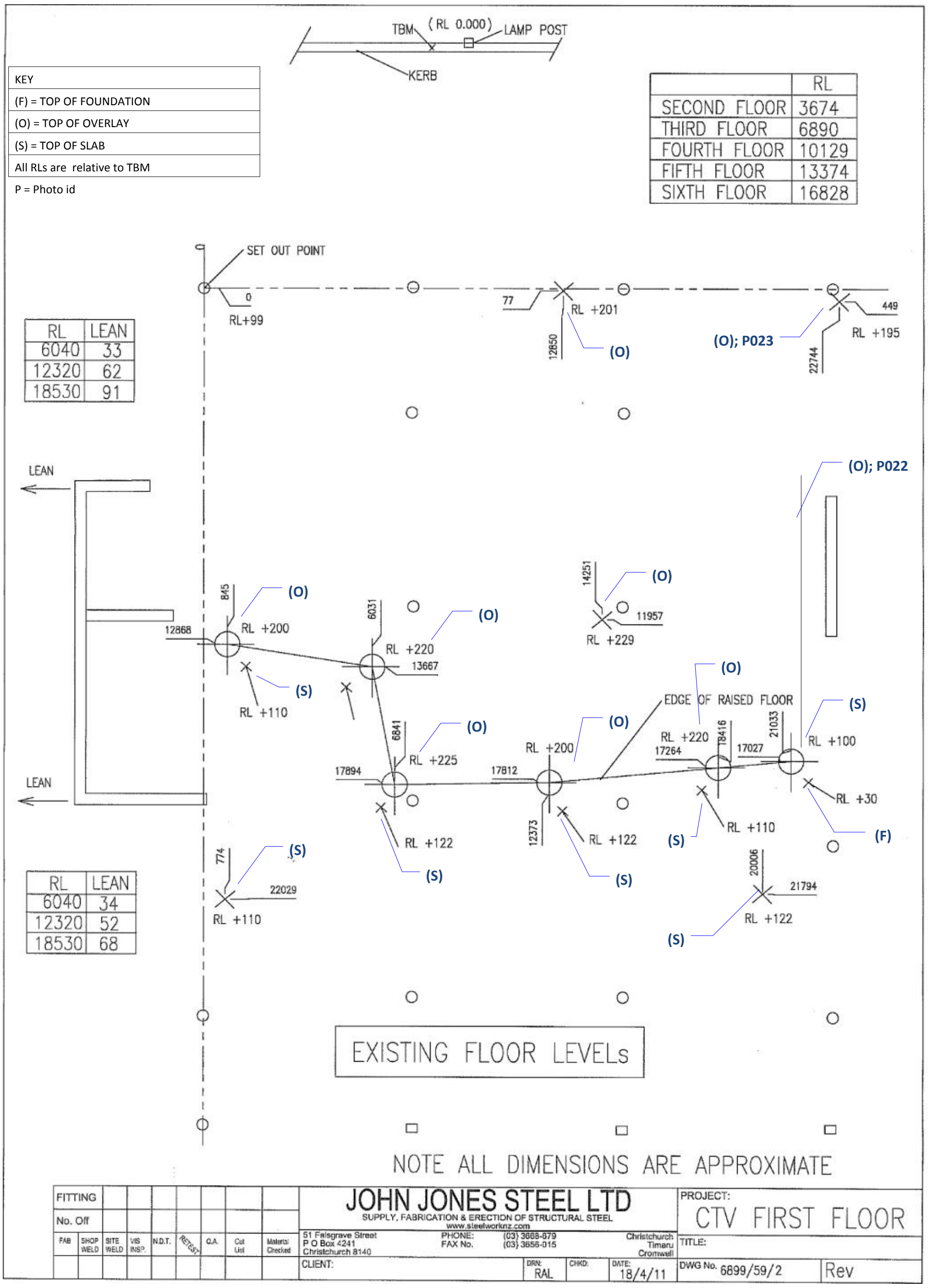

Figure 57 - JJS Dwg 2: Location of overlay edge and lift core lean annotated with levels on adjacent concrete identified as 100mm overlay (O), 125mm slab (S) or foundation beam (F)

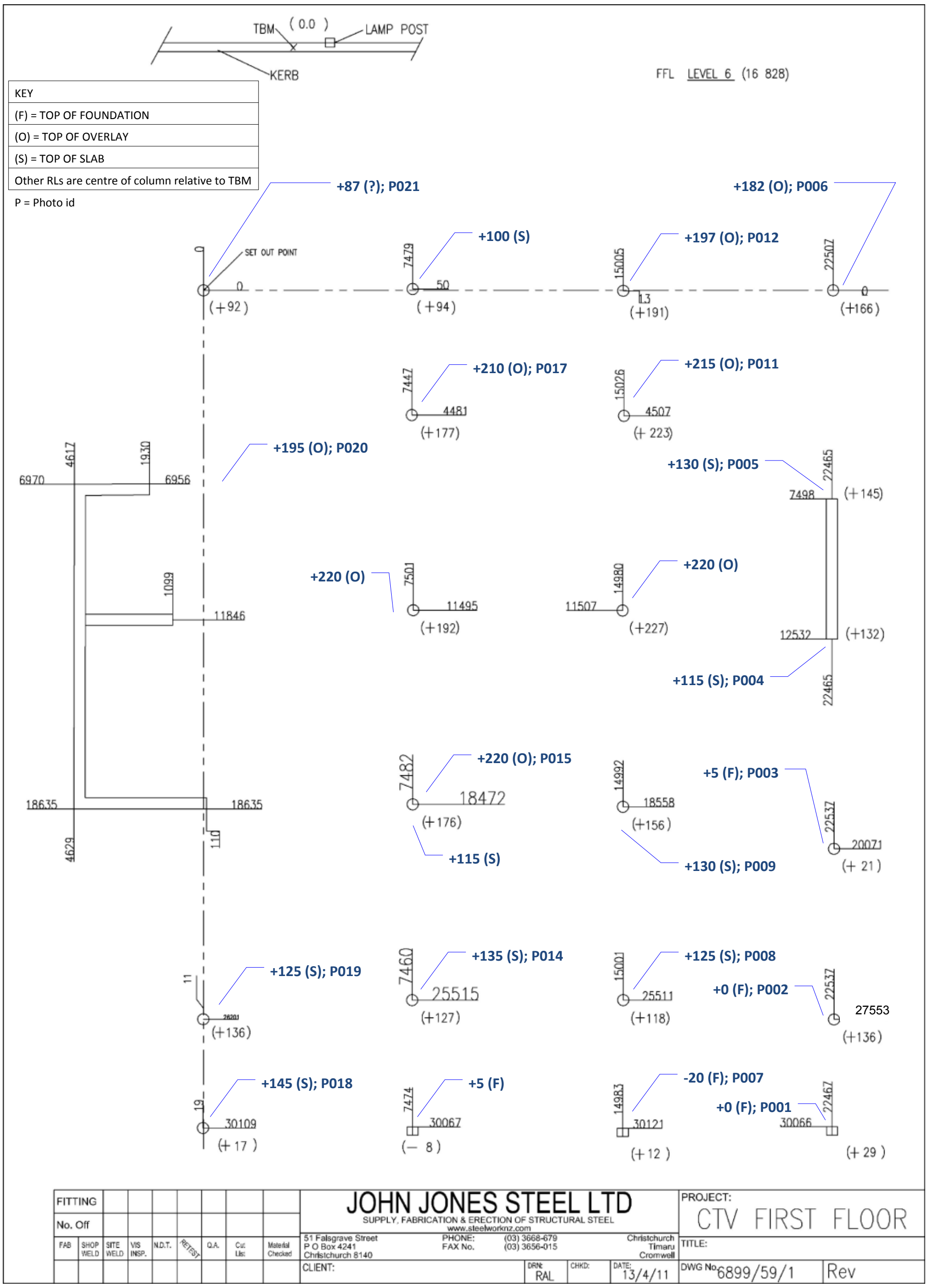

Figure 58 - JJS Dwg 1: Locations and levels at centres of demolished columns, annotated with levels on adjacent concrete identified as 100mm overlay (O), 125mm slab (S) or foundation beam (F).Photo locations are designated P###.


P001


P002


P003


P004


P005


P006


P007


P008


P009


P010


P011


P012


P013


P014


P015


P016


P017


P018

P019

P020

P021

P022

P023

APPENDIX B: FOUNDATION INSPECTION

The following are notes for the photos during the inspection by a CERA engineer in Figure 60. The location at which the photos were taken is shown in Figure 59.

A. PHOTO NOTES

P947 The floor slab exposed after removal of the overlay slab. Pavement markings indicate this area was a car park.

P948 Tops of foundation beams exposed after removal of the floor slab. The material between the beams is typical Canterbury pit-run rounded river gravel.

P968 Top of foundation beams. No damage evident. Chips are from excavator bucket.

P971 Top of foundation beam.

P960 What appears to be a foundation beam construction joint at the edge of the column pad at the south west corner of the area uncovered. There were no other joints evident in the exposed foundation beams.

P903 North side of the excavation at the northwest corner showing side of the finger beam which is founded around 1250 below the slab level on damp, firm yellow silt. The silt bearing capability was not tested but it "feels" about what one would expect for 100kPa safe pressure ground. The side of the beam still had some rotted boxing timber in place.

P964 NW corner finger beam top surface. No damage evident. Chips are from excavator bucket.

P919 Excavated south side of the finger beam showing the base slab. Water entered from a broken pipe in the side of the tower foundation. The base slab is about 650 below top of the beam.

P993 North side of the excavation at the northwest corner showing side of the finger beam shown in P903 after demolition of the core. The rotted boxing timber in P903 on the side of the footing has been removed.

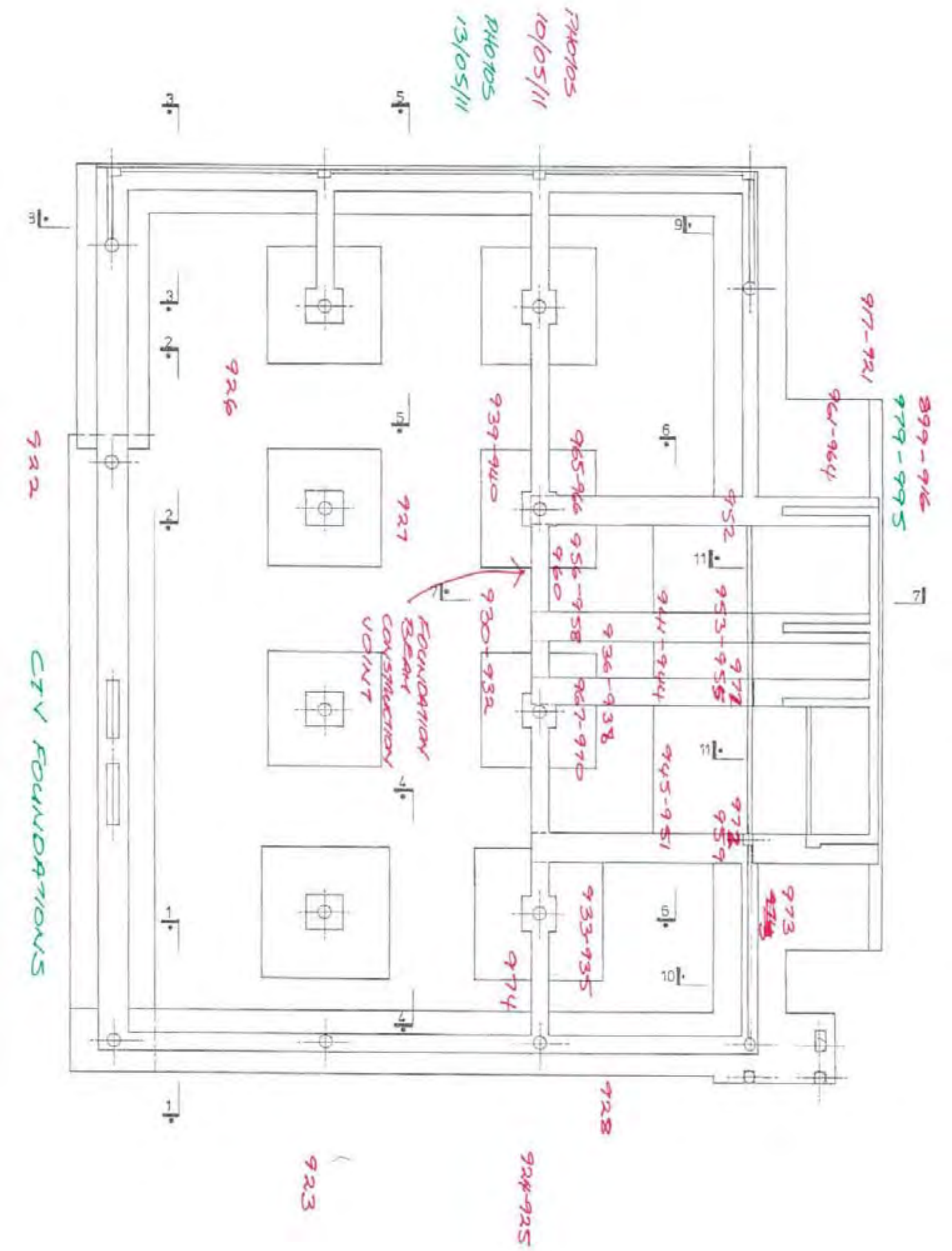

Figure 59 - Locations of photos taken during foundation inspections on 10th and 13th May, 2011 (CERA)

Figure 60 - Foundation Inspection (From left to right in rows from top) (a) P947; (b) P948; (c) P968; (d) P971; (e) P960; (f) P903; (g) P964; (h) P919; (i) P993

APPENDIX C: COLUMN STRENGTH ASSESSMENT

A. COLUMN TEST DATA AND OBSERVATIONS

CTV Columns Cores and Rebound Hammer Tests

Column ID	Test Location	Hammer Number 1	Orientation (Vertical =1200 hr)	Hammer Number 2	Orientation	Hammer Number Avg	Test Core Strength 1	Test Core Strength 2	Test Core Strength 3	Test Core Average MPa	Hammer Strength MPa	Location in Building	Failure Type and Position
400 x 400 mm Square Columns (D/E 4) (Cores Only)													
C18							25.1	12.8	13.7			At L1 at wall D/E	
							16.5	17.0	11.0	16.0			
400 mm Diameter Columns (Cores and Rebound Hammer)													
tC1	Top	42.1	1200			42.1	26.5	16.0	27.5	23.3	29.8	aka Item E25; Level 6	
tC4	Bottom	49.3	1100			49.3	47.8	45.3		46.6	44.3	Level unknown	Flexure at base and top
tC12		42	1200	46.1	1200	44.1	27.1	26.2		26.7	33.2	Level unknown; Edge	Flexure at base and mid-height
400 x 300 mm Rectangular Columns (Line A) (Cores and Rebound Hammer)													
tR6		35.7	1200	37.8	1200	36.8	24.5	26.4		25.5	22.2	Above tR7: Level unknown	Beam-column joint and mid-height
tR7		46.2	1200	46.5	1200	46.4	39.5	42.2		40.9	37.7	Below tR6; Level unknown	Mid-height and beam-column joint
tR3		35.3	1200	35.4	1200	35.4	20.5	20.1		20.3	20.5	Level unknown	Base and top
400 mm Diameter Columns (Rebound Hammer Only)										Hammer Ave MPa			
tC5	Bottom	41.1	1200			41					28.2	Level 6 to Roof	Flexure at base and mid-height
tC5	Top	38.3	1130			38				26.2	24.2		
tC6		35.6	1130			36				20.8	20.8	Level 6 to Roof; Edge	Flexure at mid-height
tC9	Top	37.6	1200			38				23.2	23.2	Level 6 to Roof; Edge	Flexure base and mid-height
tC8		33.5	1130			34				18.5	18.5	Level 5 to 6; tC1 above; Edge	Flexure at base and mid-height
tC10	Bottom	35.4	1200			35					20.6	Level 1 to 2 C21 or C22	6 D12 bars fractured at base
tC10	Top	31.1	1200			31				18.4	16.2	Downpipe in column	
tC2	Bottom	35.9	1200			36					21.2	Level 1 to 2 C23	6 D12 bars fractured at base
tC2	Top	48.1	1130			48				31.3	41.5		
tC3		41.6	1130			42				29.0	29.0	Level unknown	Flexure at base and mid-height
tC7		41.9	1200			42				29.5	29.5	Level unknown; Edge	Flexure at base and mid-height
tC11		34.3	1000			34				19.4	19.4	Level unknown; Edge	Flexure at base and mid-height
tC13		36.9	1200			37				22.4	22.4	Level unknown; Edge	Flexure at base and mid-height
400 x 300 mm Rectangular Columns (Line A) (Rebound Hammer Only)													
tR2	Bottom	40.4	1200			40					27.1	Level 6 to Roof	Beam-column joint
tR2	Top	41.6	1200			42				28.1	29.0		
tR4'	Bottom	39.6	1100			40					26.0	Level 6 to Roof	Beam-column joint
tR4'	Top	36.4	1100			36				23.9	21.8		
tR4	Bottom	45.4	1100			45					35.7	Level 5 to Level 6	Beam-column joint
tR4	Top	46.6	1100			47				37.0	38.2	tR4' above	
tR1		37.7	1130			38				23.4	23.4	Level unknown	Beam-column joint
tR5		43.6	1130			44				32.4	32.4	Level unknown	Beam-column joint and mid-height
tR8	Bottom	42.9	1200			43					31.1	Level unknown	Beam-column joint,
tR8	Top	38.9	1200			39				28.1	25.0		mid-height
tR9	Bottom	39.5	1200			40					25.8	Level unknown	Beam-column joint
tR9	Top	35.8	1200			36				23.4	21.0		
tR10		50.1	1130			50				46.3	46.3	Level unknown	
tR10'		42.9	1130			43				31.1	31.1	Level unknown	

Table 6 - Columns Core tests results, Rebound Hammer results, strengths, locations of columns and comments on failure damage. All tests were undertaken transverse to concrete casting direction.

B. CONCRETE CORE VS REBOUND HAMMER NUMBER STRENGTH RELATIONSHIP

Concrete Cores vs Hammer Numbers

Specimen	Location	Hammer Number	Orientation	Hammer Number	Orientation	Hammer Avg	Core 1 MPa	Core 2 MPa	Core 3 MPa	Core Avg MPa	Predicted MPa
tC1	Top	42.1	1200			42.1	26.5	16	27.5	27.0	29.8
tC4	Top	49.9	1200	45.9	1100	47.9	47.8	45.3		46.6	41.0
tC12	Top	46.1	1200	42.0	1200	44.1	27.1	26.2		26.7	33.3
tR3	Top	35.4	1200	35.3	1200	35.4	20.5	20.1		20.3	20.6
tR6	Top	37.8	1200	35.7	1200	36.8	24.5	26.4		25.5	22.2
tR7	Top	46.5	1200	46.2	1200	46.4	39.5	42.2		40.9	37.8

Note: tC1 core 2 has been excluded as an outlier for developing strength vs hammer number relationship

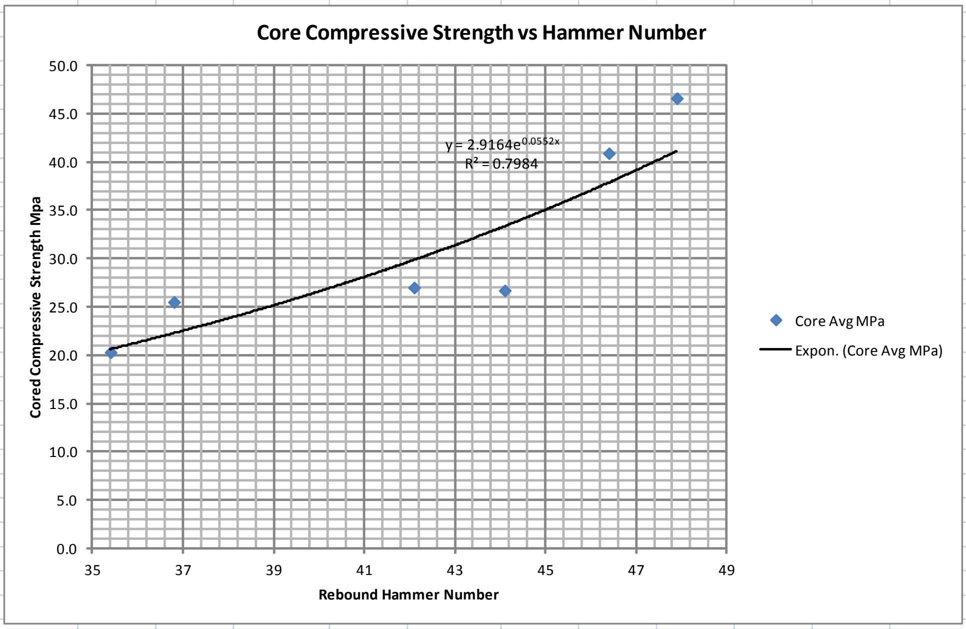

Figure 61 - Core strength to rebound hammer tests correlation curve.

C. COLUMN SCHMIDT HAMMER TESTS REPORT

(included with permission of Opus International Consultants Ltd.)

30 May 2011

Dr Clark Hyland
Director
Hyland Fatigue & Earthquake Engineering
P O Box 97282
MANUKAU 2241

OPUS

6-JHFEE.11/6LC (5884)

Dear Clark

CTV COLUMNS INSITU SCHMIDT HAMMER IMPACT INVESTIGATION

The insitu Schmidt hammer impact investigation of the CTV columns located at Burwood landfill is completed. Mr John Snook rendezvoused on site to describe the numbering requirement and locations to be tested and reported.

The following pages itemise each column tested and the horizontal location of the impacts along the column as top, middle, bottom. The impacts have been recorded in accordance with ASTM C805 and singularly recorded along with a mean achievement and other data.

I have included a Schmidt rebound chart as page 5 of 6 pages demonstrating adjustment for orientation changes as applicable. For simplicity I have stated the orientation of the test site around the columns in the manner of viewing a clock face. Hammer impacts were at all times perpendicular to the test surface.

Page 6 of 6 is the overview photograph you supplied to me in the job brief.

If I can be of additional assistance to you in any interpretation of these numbers do not hesitate to contact me.

Yours faithfully
Opus International Consultants

Geoff Jones
Laboratory Manager

Opus International Consultants Limited
Christchurch Laboratory

52C Hayton Road
Wigram, Christchurch 8042,
New Zealand

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CTV COLUMNS
BURWOOD LANDFILL
INSITU SCHMIDT HAMMER IMPACT INVESTIGATION

COLUMN:	C1	C1	C1	C2	C2	C3	C4	C4	C5	C5	C6	C7
LOCATION:	Bottom	Middle	Top	Bottom	Top		Bottom	Top	Bottom	Top		
	40	44	35	36	44	38	48	48	42	43	37	40
	44	38	-	38	50	41	51	-	36	38	-	50
	45	40	35	32	-	46	48	47	42	32	37	38
	-	46	46	32	48	38	52	44	41	-	32	48
IMPACT:	38	39	50	34	44	39	48	49	41	38	32	36
	39	44	42	38	45	45	49	48	38	39	38	36
	39	39	45	44	55	45	49	40	44	42	40	-
	42	38	38	33	51	40	49	42	-	38	38	41
	38	44	44	32	48	40	49	47	44	37	32	47
	38	36	44	40	-	44	50	48	42	38	34	41
AVERAGE IMPACT:	40.3	40.8	42.1	35.9	48.1	41.6	49.3	45.9	41.1	38.3	35.6	41.9
ORIENTATION:	1200	1200	1200	1200	1130	1130	1100	1100	1200	1130	1130	1200

SCHMIDT HAMMER IDENTIFICATION: 4-020
DATE VALIDATED: 7/10
DATE DUE: 7/11
ESTIMATED AMBIENT TEMPERATURE: 15°C
COLUMN CONFIGURATION: 400 MM ϕ

HAMMER IMPACT ANGLE: 90° TO SURFACE
TEST SURFACE PREPARATION: CLEANED UNGROUND
TEST SURFACE CONDITION: DRY
DASHED LINE: READING > 6 UNITS FROM MEAN
TIME OF TEST SERIES: 0900-1200

Lab Ref: 5884
Page 2 of 6 pages

CTV COLUMNS
BURWOOD LANDFILL
INSITU SCHMIDT HAMMER IMPACT INVESTIGATION

COLUMN:	C8	C9	C10	C10	C11	C12	C13	R1	R2	R2	R3	R4
LOCATION:		Top	Bottom	Top					Bottom	Top		Bottom
	28	32	32	36	30	42	37	37	43	42	36	46
	30	28	38	31	34	43	36	36	44	46	39	45
	34	34	39	31	32	43	34	35	35	38	35	-
	36	41	-	34	40	47	40	41	40	48	34	52
IMPACT:	34	40	36	28	34	41	41	38	41	36	34	46
	40	41	42	27	36	38	38	39	42	40	36	42
	35	38	32	34	32	42	36	38	38	42	35	44
	28	42	32	26	36	38	37	36	44	44	34	44
	32	42	32	30	38	44	32	39	39	38	34	48
	38	38	36	34	31	42	38	38	38	42	36	42
AVERAGE IMPACT:	33.5	37.6	35.4	31.1	34.3	42.0	36.9	37.7	40.4	41.6	35.3	45.4
ORIENTATION:	1130	1200	1200	1200	1000	1200	1200	1130	1200	1200	1200	1100

SCHMIDT HAMMER IDENTIFICATION: 4-020
DATE VALIDATED: 7/10
DATE DUE: 7/11
ESTIMATED AMBIENT TEMPERATURE: 15°C
COLUMN CONFIGURATION: 400 MM ϕ

HAMMER IMPACT ANGLE: 90° TO SURFACE
TEST SURFACE PREPARATION: CLEANED UNGROUND
TEST SURFACE CONDITION: DRY
DASHED LINE: READING > 6 UNITS FROM MEAN
TIME OF TEST SERIES: 0900-1200

Lab Ref: 5884
Page 3 of 6 pages

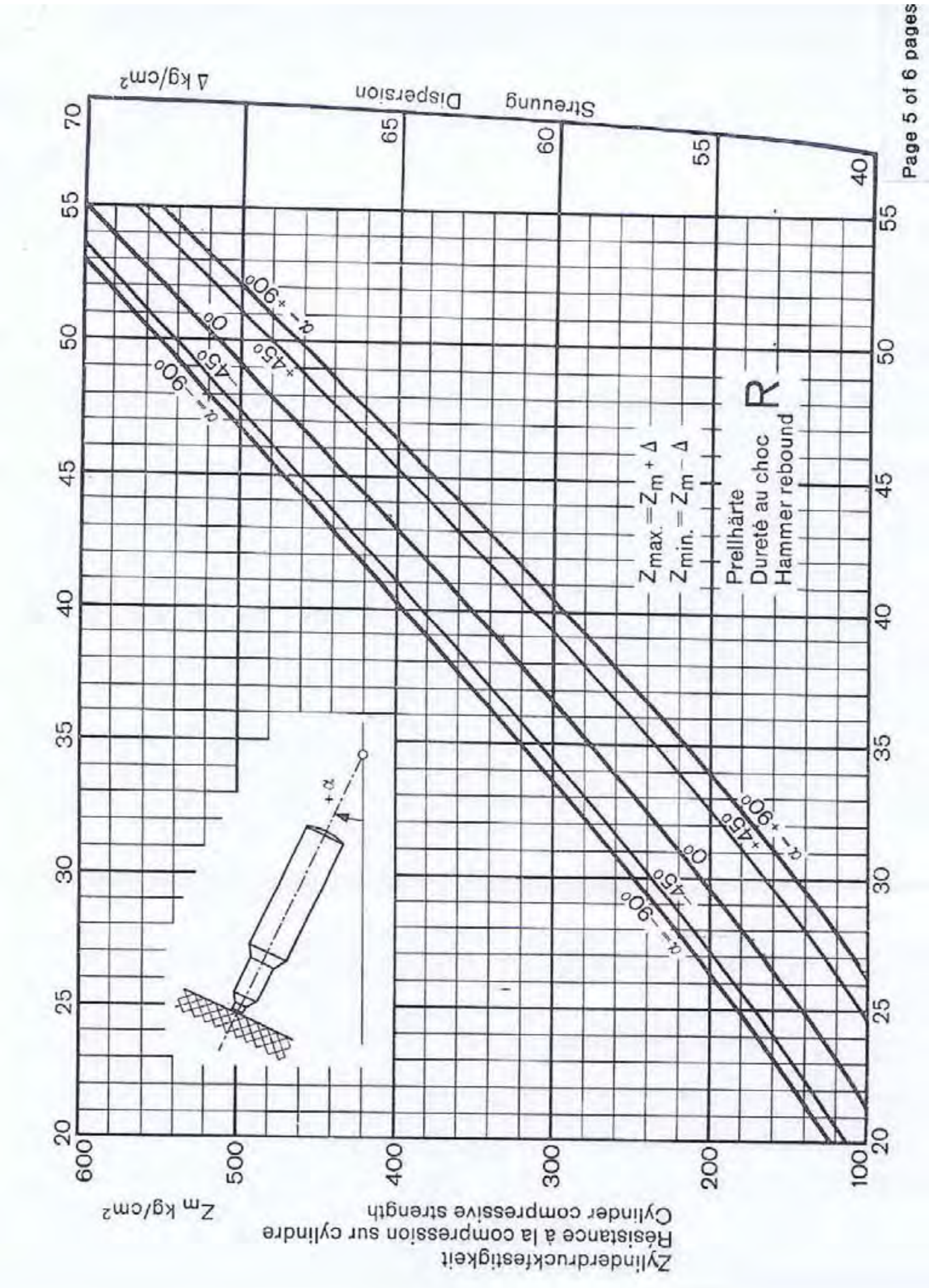
CTV COLUMNS
BURWOOD LANDFILL
INSITU SCHMIDT HAMMER IMPACT INVESTIGATION

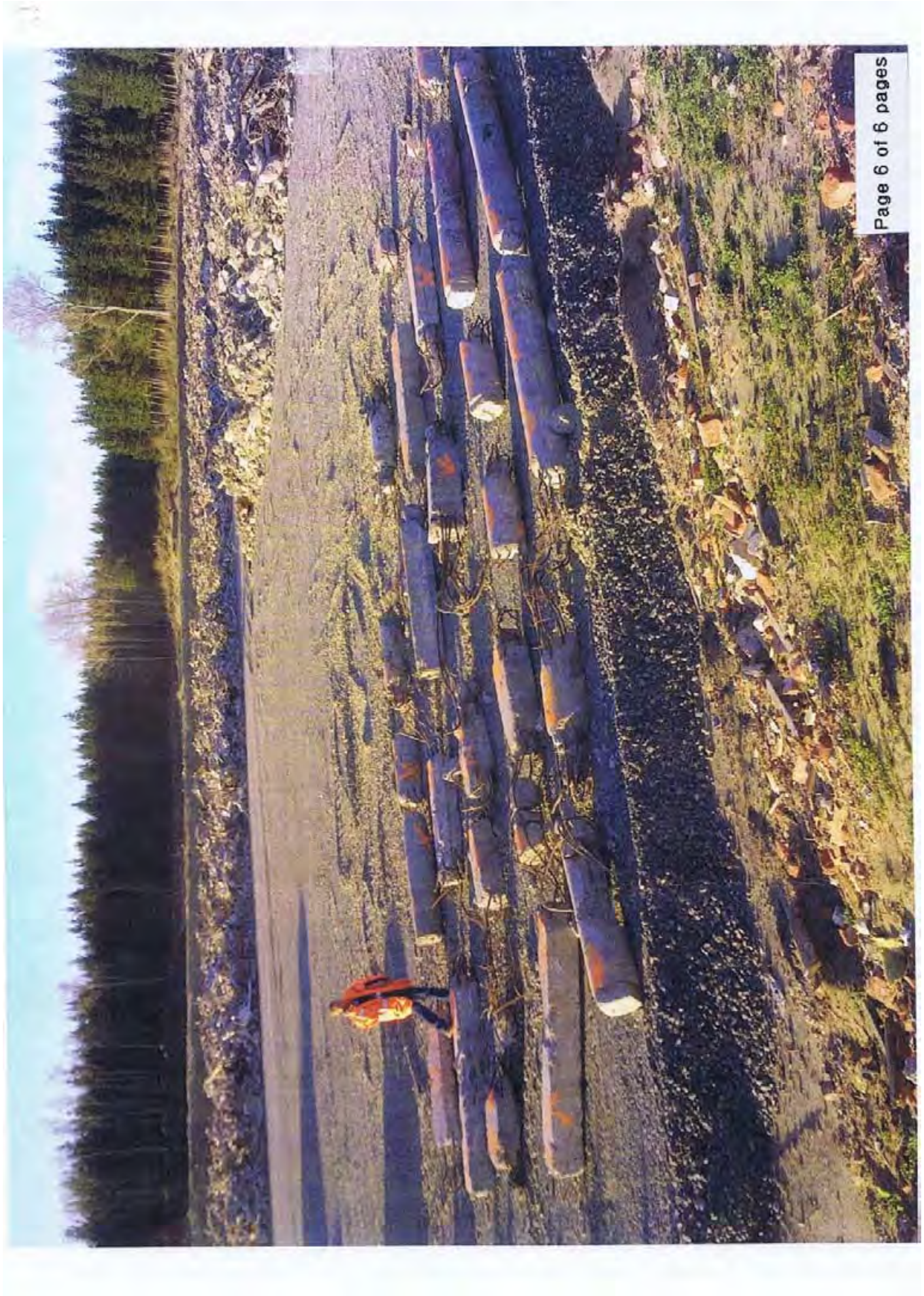
COLUMN:	R4	R4'	R4'	R5	R6	R7	R8	R8	R9	R9	R10	R10'
LOCATION:	Top	Bottom	Top				Bottom	Top	Bottom	Top		
	49	38	38	42	40	46	46	38	41	-	50	44
	49	42	40	44	-	45	46	39	36	34	51	48
	49	42	38	50	30	46	46	41	46	38	52	40
	44	38	38	41	32	42	39	36	40	37	50	41
IMPACT:	40	43	36	46	38	47	42	36	41	33	46	38
	44	42	36	43	40	47	38	39	40	36	48	-
	46	35	32	38	37	46	40	42	36	38	51	40
	-	34	32	46	36	48	42	41	39	38	49	45
	48	44	36	46	32	48	47	36	38	34	52	44
	50	38	38	40	36	47	43	41	38	34	52	46
AVERAGE IMPACT:	46.6	39.6	36.4	43.6	35.7	46.2	42.9	38.9	39.5	35.8	50.1	42.9
ORIENTATION:	1100	1100	1100	1130	1200	1200	1200	1200	1200	1200	1130	1130

SCHMIDT HAMMER IDENTIFICATION: 4-020
DATE VALIDATED: 7/10
DATE DUE: 7/11
ESTIMATED AMBIENT TEMPERATURE: 15°C
COLUMN CONFIGURATION: 400 MM ф

HAMMER IMPACT ANGLE: 90° TO SURFACE
TEST SURFACE PREPARATION: CLEANED UNGROUND
TEST SURFACE CONDITION: DRY
DASHED LINE: READING > 6 UNITS FROM MEAN
TIME OF TEST SERIES: 0900-1200

Lab Ref: 5884
Page 4 of 6 pages


Δ kg/cm²
Dispersion
Streuung
Zylinderdruckfestigkeit
Résistance à la compression sur cylindre
Cylinder compressive strength
Zm kg/cm²
Zmax. = Zm + Δ
Zmin. = Zm − Δ
Prellhärte
Dureté au choc
Hammer rebound
R
α = +90°
+45°
0°
−45°
α = −90°
Page 5 of 6 pages


Page 6 of 6 pages

D. COLUMNS SCHMIDT HAMMER CORES TESTS

(Included with permission of Opus International Consultants Ltd.

10 June 2011

Dr Clark Hyland
Director
Hyland Fatigue & Earthquake Engineering
P O Box 97282
MANUKAU 2241

6-JHFEE.11/6LC (5907)

Dear Clark

CTV COLUMNS SCHMIDT HAMMER IMPACT RECOVERED CORE COMPARISON

The core recovery programme with subsequent compression test and insitu Schmidt hammer impact comparison of the CTV columns located at Burwood landfill is completed.

Testing with the Schmidt hammer was undertaken in accordance with standard test method for rebound number of hardened concrete, ASTM C805M-08, clause 5.2. The five selected locations are itemised with average impact values and the stated orientation of each set. I have also presented the compressive strength of the duplicate cores removed from the corresponding Schmidt impact site. Once again I attach the inferred compressive strength versus Schmidt impact sheet, but I leave it to you to formulate the relationship from the achieved results. Multiplying kg/cm² by 0.098 obtains inferred MPa.

If I can be of additional assistance to you in any interpretation of these numbers do not hesitate to contact me.

Yours faithfully
Opus International Consultants

Geoff Jones
Laboratory Manager

Opus International Consultants Limited
Christchurch Laboratory

52C Hayton Road
Wigram, Christchurch 8042,
New Zealand

Telephone: +64 3 343 0739
Facsimile: +64 3 343 0737
Website: www.opus.co.nz

CONCRETE COMPRESSION OF CORES TEST REPORT

Project :	**Material Strength Investigation**
Location :	**Canterbury Television Building, Christchurch**
Client :	**Hyland Fatigue & Earthquake Engineering Limited**
Contractor :	**Opus International Consultants Laboratory**
Sampled by :	**R Jones & W Parsons**
Date sampled :	**8 June 2011**
Sampling method :	**Concrete Hole Saw**
Sample description :	**Drilled Concrete Core**
Sample condition :	**Dry as received**
Date cored :	**8 June 2011**
Source of concrete :	**Insitu 400mm Diameter Columns**
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 :	5907
Client Ref No :	C4 & C12

Test Results

Lab reference no		160 (1)	160 (2)	160 (3)	160 (4)
Client reference no		C4 1	C4 2	C12 1	C12 2
Date tested		9 June 2011			
Dry cured	(days)	1			
Size & position of any reinforcement		No Steel			
Visual description		Standard Core			
Average core diameter	(mm)	68.9	69.2	69.0	69.1
Average core length	(mm)	135.3	136.5	140.6	83.0
Density	(kg/m^3)	2412	2433	2378	2385
Height diameter ratio		1.97	1.97	2.04	1.20
Conditioning		Dry			
Load at failure	(kN)	177.9	171.7	100.5	114.8
Compressive strength	(MPa)	48.0	45.5	27.0	30.5
Compressive strength (Factor D adjustment)	(MPa)	47.8	45.3	27.1	26.2
Type of fracture		Not established			

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 : 9 June 2011
Date reported : 9 June 2011

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IANZ Approved Signatory

Designation : *Laboratory Manager*
Date : 9 June 2011

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CONCRETE COMPRESSION OF CORES TEST REPORT

Project :	**Material Strength Investigation**
Location :	**Canterbury Television Building, Christchurch**
Client :	**Hyland Fatigue & Earthquake Engineering Limited**
Contractor :	**Opus International Consultants Laboratory**
Sampled by :	**R Jones & W Parsons**
Date sampled :	**8 June 2011**
Sampling method :	**Concrete Hole Saw**
Sample description :	**Drilled Concrete Core**
Sample condition :	**Dry as received**
Date cored :	**8 June 2011**
Source of concrete :	**Insitu 400mm x 300mm Columns**
Grade of concrete :	**Not Advised**
Design strength :	**Not Advised**
Actual slump :	**Not Advised**
Date laid :	**Not Advised**

Project No :	6-HFEE.11/006LC
Lab Ref No :	5907
Client Ref No :	R3, R6 & R7

Test Results

Lab reference no	160 (6)	160 (7)	160 (8)	160 (9)	160 (10)	160 (11)
Client reference no	**R3 1**	**R3 2**	**R6 1**	**R6 2**	**R7 1**	**R7 2**
Date tested	9 June 2011					
Dry cured (days)	1					
Size & position of any reinforcement	No Steel					
Visual description	Standard Core					
Average core diameter (mm)	68.8	68.7	68.7	69.0	68.8	68.8
Average core length (mm)	138.3	140.2	137.9	135.9	137.3	140.4
Density (kg/m^3)	**2259**	**2234**	**2388**	**2385**	**2356**	**2347**
Height diameter ratio	2.01	2.04	2.01	1.97	2.00	2.04
Conditioning	Dry					
Load at failure (kN)	76.5	73.8	91.6	98.8	146.8	155.7
Compressive strength (MPa)	20.5	20.0	24.5	26.5	39.5	42.0
Compressive strength (Factor D adjustment) (MPa)	**20.5**	**20.1**	**24.5**	**26.4**	**39.5**	**42.2**
Type of fracture	Not established					

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 : 9 June 2011
Date reported : 9 June 2011

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Designation : *Laboratory Manager*
Date : 9 June 2011

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CONCRETE COMPRESSION OF CORES TEST REPORT

Project :	**Material Strength Investigation**
Location :	**Canterbury Television Building, Christchurch**
Client :	**Hyland Fatigue & Earthquake Engineering Limited**
Contractor :	**Opus International Consultants Laboratory**
Sampled by :	**R Jones & W Parsons**
Date sampled :	**8 June 2011**
Sampling method :	**Concrete Hole Saw**
Sample description :	**Drilled Concrete Core**
Sample condition :	**Dry as received**
Date cored :	**8 June 2011**
Source of concrete :	**Insitu 400mm x 300mm Columns**
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 :	5907
Client Ref No :	R3, R6 & R7

Test Results

Lab reference no		160 (6)	160 (7)	160 (8)	160 (9)	160 (10)	160 (11)
Client reference no		**R3 1**	**R3 2**	**R6 1**	**R6 2**	**R7 1**	**R7 2**
Date tested		9 June 2011					
Dry cured	(days)	1					
Size & position of any reinforcement		No Steel					
Visual description		Standard Core					
Average core diameter	(mm)	68.8	68.7	68.7	69.0	68.8	68.8
Average core length	(mm)	138.3	140.2	137.9	135.9	137.3	140.4
Density	(kg/m^3)	**2259**	**2234**	**2388**	**2385**	**2356**	**2347**
Height diameter ratio		2.01	2.04	2.01	1.97	2.00	2.04
Conditioning		Dry					
Load at failure	(kN)	76.5	73.8	91.6	98.8	146.8	155.7
Compressive strength	(MPa)	20.5	20.0	24.5	26.5	39.5	42.0
Compressive strength (Factor D adjustment)	(MPa)	**20.5**	**20.1**	**24.5**	**26.4**	**39.5**	**42.2**
Type of fracture		Not established					

Test Methods	Notes
Testing of Cores, NZS 3112 : Part 2 : 1986, Clause 9	Sampling is outside the laboratory's scope of accreditation
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)	

Date tested : 9 June 2011
Date reported : 9 June 2011

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IANZ Approved Signatory

Designation : *Laboratory Manager*
Date : 9 June 2011

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

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E. LEVEL 1 400 SQUARE C18 AND LEVEL 6 400 CIRCULAR COLUMNS

Two sets of compression tests were undertaken on concrete extracted from the 400 mm square C18 column stub at Level 1. The cores were extracted in such a way as to seek to avoid any effect of fire on the concrete properties.

Three cores were taken from column Item E25 which was identified as a Level 6 column. It was also found amongst the Burwood Landfill columns extracted and designated tC1 for the Rebound (Schmidt) Hammer testing.

i. Set 1 Results

CONCRETE COMPRESSION OF CORES TEST REPORT

Project :	**Material Strength Investigation**
Location :	**Canterbury Television Building, Christchurch**
Client :	**Hyland Fatigue & Earthquake Engineering Limited**
Contractor :	**Concut Limited**
Sampled by :	**Concut Limited (John)**
Date sampled :	**28 April 2011**
Sampling method :	**Concrete Hole Saw**
Sample description :	**Drilled Concrete Core**
Sample condition :	**Dry as received**
Date cored :	**28 April 2011**
Source of concrete :	**Insitu Square 400mm Column**
Grade of concrete :	**35 MPa**
Design strength :	**35 MPa**
Actual slump :	**Not Advised**
Date laid :	**Not Advised**

Project No :	6-HFEE.11/006LC
Lab Ref No :	5825
Client Ref No :	Clark Hyland

Test Results

Lab reference no		124 (1)	124 (2)	124 (3)
Client reference no		CTV 1	CTV 2	CTV 3
Date tested			5 April 2011	
Dry cured	(days)		7	
Size & position of any reinforcement		No Steel	No Steel	No Steel
Visual description		Standard Core	Standard Core	Standard Core
Average core diameter	(mm)	67.7	67.7	67.7
Average core length	(mm)	79.0	81.3	44.2
Density	(kg/m^3)	**2350**	**2330**	**2340**
Height diameter ratio		1.17	1.20	0.65
Conditioning			Dry	
Load at failure	(kN)	106.8	62.3	53.4
Compressive strength	(MPa)	29.5	17.5	15.0
Compressive strength (Factor D adjusti	(MPa)	**25.1**	**12.8**	**13.7**
Type of fracture		Not established	Not established	Not established

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 : 5 May 2011
Date reported : 5 May 2011

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IANZ Approved Signatory

Designation : *Laboratory Manager*
Date : 5 May 2011

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

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ii. Set 2 Results and Specimen Examination Report

11 May 2011

Mr Clark Hyland
Director
Hyland Fatigue & Earthquake Engineering
P O Box 97282
MANUKAU 2241

6-JHFEE.11/6LC (5833)

Dear Clark

CTV CONCRETE CORE RECOVERY & COMPRESSION TESTING

The core recovery visit to Canterbury Television site is completed in accordance with your brief to this laboratory of 5 May 2011.

Two 100mm diameter cores were successfully recovered while a third fragmented in the barrel during extrusion. The two recoveries were later deemed unsuitable for compression testing due to significant rebar within the cores and being too short for realistic 2:1 testing configuration. Upon completion of our coring I requested the Dormer Contracting operator to scissor snip the entire column plinth at it's base whereby we were able to recover suitably sized fragments for coring upon return to the laboratory.

We extruded three nominal 69 mm diameter cores and had sufficient length within each fragment to undertake compression testing utilising the D factor height to diameter ratio adjustment on one core only.

Core CTV1: With the scissor snip tool pulling the rebars apart for us to gain sample access, I am unable to determine the orientation of the core sample. Looking at the rebar imprint size I am assuming it was a vertical member and therefore we have cored horizontally. There are no smooth sides on this sample inferring it has come from central column around 1.0 – 1.5 metres above ground level. There is no scorching apparent.

Core CTV2: With the rebar size deformation on the fragment it is inferred we have recovered a horizontal core again. This fragment is an inner piece of the column because there are no edges visible. A minor observation is that there appears to be a significant 13mm aggregate fraction throughout this core interspersed amongst the 19mm. No scorching is apparent. As with all three fragments, recovery was 1.0 – 1.5 metres above ground level.

Core CTV3: By orientation of the principal rebar imprints we have cored vertically in this fragment. An outside surface is apparent and the core has been recovered 25mm from this surface. There is significant scorched sooting over this face, and minor scorching around a corner from the principal face. The core is unaffected.

I have included snap shots of the cored fragments and the cores prior to sizing and compression testing. There may be something of interest for you in viewing these. We have retained the cores also for your perusal.

If I may be of additional assistance to you do not hesitate to contact me.

Opus International Consultants Limited
Christchurch Laboratory

52C Hayton Road
Wigram, Christchurch 8042,
New Zealand

Telephone: +64 3 343 0739
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Website: www.opus.co.nz

Yours faithfully
Opus International Consultants

Geoff Jones
Laboratory Manager

CONCRETE COMPRESSION OF CORES TEST REPORT

OPUS

Project :	**Material Strength Investigation**
Location :	**Canterbury Television Building, Christchurch**
Client :	**Hyland Fatigue & Earthquake Engineering Limited**
Contractor :	**Opus International Consultants Laboratory**
Sampled by :	**G & R Jones**
Date sampled :	**10 May 2011**
Sampling method :	**Concrete Hole Saw**
Sample description :	**Drilled Concrete Core**
Sample condition :	**Dry as received**
Date cored :	**10 May 2011**
Source of concrete :	**Insitu Square 400mm Column**
Grade of concrete :	**35 MPa**
Design strength :	**35 MPa**
Actual slump :	**Not Advised**
Date laid :	**Not Advised**

Project No :	**6-HFEE.11/006LC**
Lab Ref No :	**5833**
Client Ref No :	Clark Hyland

Test Results

Lab reference no		131 (1)	131 (2)	131 (3)
Client reference no		**CTV 1**	**CTV 2**	**CTV 3**
Date tested			11 April 2011	
Dry cured	(days)		1	
Size & position of any reinforcement		No Steel	No Steel	No Steel
Visual description		Standard Core	Standard Core	Standard Core
Average core diameter	(mm)	69.2	69.0	69.0
Average core length	(mm)	137.0	100.7	141.0
Density	(kg/m^3)	**2310**	**2330**	**2360**
Height diameter ratio		1.98	1.46	2.04
Conditioning			Dry	
Load at failure	(kN)	62.3	69.4	40.9
Compressive strength	(MPa)	16.5	18.5	11.0
Compressive strength (Factor D adjusti	(MPa)	**16.5**	**17.0**	**11.0**
Type of fracture		Not established	Not established	Not established

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 : 11 May 2011
Date reported : 11 May 2011

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

Designation : *Laboratory Manager*
Date : 11 May 2011

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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Christchurch Laboratory
Quality Management Systems Certified to ISO 9001

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CTV 1


CTV 2


CTV 3


SCORCHED
SCORCHED

CONCRETE COMPRESSION OF CORES TEST REPORT

Project :	**Material Strength Investigation**
Location :	**CTV Building, Christchurch**
Client :	**Hyland Fatigue & Earthquake Engineering Limited**
Contractor :	**Concut Limited**
Sampled by :	**Concut Limited (John)**
Date sampled :	**12 March 2011**
Sampling method :	**Concrete Hole Saw (Horizontal)**
Sample description :	**Drilled Concrete Core**
Sample condition :	**Damp as received**
Date cored :	**12 March 2011**
Source of concrete :	**CTV Building, 400Ø Column**
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 :	5673
Client Ref No :	Clark Hyland

Test Results

Lab reference no		055	055	055
Client reference no			**CTV 400Ø Column**	
Date tested		28/03/11	28/03/11	28/03/11
Dry cured	(days)	7	7	7
Size & position of any reinforcement		No Steel	No Steel	19mm & 6mm re-bar
Visual description		Horizontal Core	Horizontal Core	Horizontal Core
Average core diameter	(mm)	95.8	96.0	95.9
Average core length	(mm)	190.9	194.5	194.6
Density	(kg/m^3)	**2324**	**2331**	**2443**
Height diameter ratio		1.99	2.03	2.03
Conditioning		Dry	Dry	Dry
Load at failure	(kN)	189.5	116.5	198.4
Compressive strength	(MPa)	**26.5**	**16.0**	**27.5**
Type of fracture		Cone/Shear	Shear	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

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APPENDIX D: REINFORCING STEEL, DRAG BAR ANCHORS AND DECKING TEST RESULTS

Reinforcing steel and profiled metal deck samples were tested by SAI Global Ltd in Christchurch. (Test report is included in full with permission of SAI Global Ltd).

Drag Bar threaded anchor rod hardness tests were undertaken by Materials and Testing Laboratories Ltd, Auckland ("MTL"). (Test report is included in full with permission of MTL).

SAI GLOBAL

Date of Issue: 17 March 2011
Reference: P5665
Page 1 of 6 Pages

TEST REPORT

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

Attention: Dr Clark Hyland

CUSTOMER REFERENCE: Dr Clark Hyland – CTV 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) D16 reinforcing bar samples, (E1)
Two (2) D28 reinforcing bar samples, (E1E, E1W)
One (1) D24 reinforcing bar samples, (E3)
Two (2) D12 reinforcing bar samples, (E4)
Three (3) D24 reinforcing bar samples, (E4)
Two (2) R6 reinforcing bar samples from mesh,
Three (3) 0.8mm galvanized sheet metal samples.

Three galvanised sheet metal samples extracted from formwork

DATE OF TEST: 15 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

This Laboratory is registered by the Testing Laboratory Registration Council of New Zealand. The tests reported herein have been performed in accordance with its terms of registration. This report may not be reproduced except in full. Laboratory Registration Number: 197

SAI Global (NZ) Ltd 52 Hayton Road P O Box 6178 Christchurch 8442 New Zealand Tel: +64 3 961 6090

Imtest Group of Laboratories, part of SAI Global

Date of Issue: 17 March 2011
Reference: P5665
Page 2 of 6 Pages

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.
Three sheet metal samples used for concrete formwork were also supplied for determination of their mechanical properties.
Tensile tests were performed in accordance with AS1391 on all of the supplied samples and percentage elongation measurements in accordance with ISO 15630-1 were performed on the reinforcing steel.
The sample markings on the reinforcing bars supplied are shown in figures 1 and 2.
The markings of the D16 sample are shown in figure 2 and the markings of all other deformed bars are shown in figure 1.

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.

Date of Issue: 17 March 2011
Reference: P5665
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Test Results

Mechanical Properties

Sample Identification	Size	Measured Diameter (mm)	Elongation at Maximum Force Agt (%)	Yield Stress Re, (MPa) *Rp0.2 (MPa)	Ultimate Tensile Stress Rm,	Ratio Rm/Re
E1a	D16	15.21	15.6	447	595	1.33
E1b	D16	15.14	17.8	451	596	1.32
E1c	D16	15.16	15.6	453	595	1.31
E1 W	D28	26.80	16.8	447	612	1.37
E1 E	D28	26.99	13.5	464	627	1.35
E3	D24	22.80	17.1	445	609	1.37
E4a	D12	11.46	17.0	518	677	1.31
E4b	D12	11.42	15.0	518	677	1.31
E4a	D24	22.97	16.6	444	607	1.37
E4b	D24	22.84	17.9	449	608	1.36
E4c	D24	22.85	17.2	445	603	1.36
Mesh a	R6	5.98	3.8	*617	666	1.08
Mesh b	R6	5.98	4.5	*614	664	1.08

Table 1

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)
E1a	D16	1.02	0.97	0	10.0	0.10
E1b	D16	1.19	0.98	0	10.0	0.11
E1c	D16	1.00	0.99	0	10.0	0.10
E1 W	D28	1.71	1.70	0	16.7	0.10
E1 E	D28	1.75	1.63	0	16.9	0.10
E3	D24	1.46	1.27	0	16.2	0.08
E4a	D12	0.73	0.67	0	7.8	0.09
E4b	D12	0.77	0.68	0	7.8	0.09
E4a	D24	1.47	1.47	0	16.1	0.09
E4b	D24	1.50	1.34	0	16.1	0.09
E4c	D24	1.53	1.51	0	16.2	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 – All other deformed bars

Figure 2 – D16

TEST RESULTS

Reference: P5665

Date: 17 March 2011

Page 6 of 6 pages

TEST EQUIPMENT USED: Avery Universal Test Machine (IMT233)
Digital Micrometer (IMT No.: 027)
Digital Calipers (IMT No.: 741)
Epsilon Extensometer (IMT No.: 693)

Material Designation	Sample Number	Mean Dimensions:		Mean Cross Sectional Area (mm2)	Gauge Length (mm)	Yield Load (kN)	Elongation Length (mm)	Tensile Load (kN)	Percent Elongation (%)	Tensile Strength (MPa)	Yield Strength (MPa)*
		Mean Thickness: (mm)	Mean Width: (mm)								
Steel	CTV a	0.816	20.02	16.34	80	9.88	82.9	9.88	3.5	605	605
Steel	CTV b	0.805	20.07	16.16	80	9.97	82.9	9.97	3.5	617	617
Steel	CTV c	0.812	20.00	16.24	80	10.23	83.2	10.23	4.0	630	630

Method used to determine Yield strength: Upper yeild point

Comments: The lower yield point was not recorded as the sample yielded outside the span of the extensometer.
The elongation result should be used as an indication only as the fracture surface was within one third of the original gauge length to the nearest gauge mark.
There was visible signs of heat/burning on one side of the samples

SAI GLOBAL

Annexed to SAI Global (NZ) Ltd Report Number P5665
Page 1 of 1

SAI GLOBAL (NZ) LIMITED

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Imtest Group of Laboratories, part of SAI Global

/ CONTINUED

MATERIALS & TESTING LABORATORIES LIMITED

Destructive & Non-destructive Test House

CERTIFICATE OF TEST

Report No. 33178/1
Page 1 of 1 page

Reference: **HYLAND ENGINEERING**
Attention: Dr Clark Hyland

Hardness Rockwell "B" Scale

Test Equipment: Wilson Rockwell Hardness Tester, Model 4JR, Serial No. 4JR 1177.
Calibration due October 2011.

Sample Tested: M20 Threaded Rods

RESULTS:

	Galvanized	Not Galvanized
1	85	93
2	85	92
3	86	93
Average	**85RB**	**93RB**

Tested by: AG
Date tested: 31/08/11
Reported by: AG
Date reported: 31/08/11
Checked by:

Signed by: ....
A. GHERMAN

P.O. BOX 14-042, PANMURE 1741, 172g MARUA RD, MT WELLINGTON, AUCKLAND, N.Z. TEL: 579 0262 FAX: 579 0260 Email: admin@mtlabs.co.nz

Figure 62 - Drag Bar slab threaded anchor rod hardness test results

APPENDIX E: WALL, BEAM AND SLAB CONCRETE CORES TEST RESULTS

Concrete cores were cut from the line 1 South Wall element marked E4; The lower portion of the Line 5 wall at the stair well on the North Core; in a precast log beam and into two portions of suspended slab still attached to concrete beams.

Testing was undertaken by the Christchurch laboratory of OPUS International Consultants Ltd in conjunction with their Wellington laboratory which undertook Modulus of elasticity tests and compressive strength tests on the samples extracted from the Line 1 and 5 shear walls.

(Test reports included with permission of Opus International Consultants Ltd)

A. LINE 1 LEVEL 4 SOUTH WALL: E4 COMPRESSIVE STRENGTH

CONCRETE CORE COMPRESSION TEST REPORT

OPUS INTERNATIONAL CONSULTANTS

Hyland Fatigue & Earthquake Engineering Ltd
P.O. Box 97282
Manukau
Auckland 2241

Attenion: Clark Hyland

Project :	**CTV Building**
Location :	**E4: Line 1 Shearwall**
Client :	**Hyland Fatigue & Earthquake Engineering Ltd**
Contractor :	-
Sampled by :	**Concut**
Date poured :	-
Sampling method :	**Core**
Grade :	-
Actual slump (mm) :	-
Specified slump (mm) :	-
Date received :	**17/03/11**

Project No :	**6JHFEE.11/6CL**
Lab Ref No :	**4-11/027**
Client Ref No :	-

Test Results

Date tested		25/03/11			
Age at test	(days)	-			
Sample ID		E4(1)	E4(2) A	E4(2) B	E4(3)
Average diameter	(mm)	95.7	95.7	95.6	95.6
Length	(mm)	192.0	194.0	172.0	191.5
Length to diameter ratio		2.01	2.03	1.80	2.00
Compressive strength	(MPa)	30.0	33.0	-	34.0
Corrected compressive strength [1]	(MPa)	-	-	31.0	-
Ave compressive strength	**(MPa)**	**32.0**			
Ends capped		2	2	2	2
Defects prior to testing		-	-	-	-
Failure mode		Normal	Normal	Normal	Normal
Conditioning type [2]		Dry	Dry	Dry	Dry

Test Methods	Notes
Compressive strength: NZS 3112:Part 2: 1986 Clause 9 Capping: NZS 3112 : 1986, Pt 2 Clause 4 (amendment No 2 2000) Correction for length to diameter ratio: in-house method CL-04-511.	1. Strengths of specimens with length to diameter ratio 1.00 to 1.90 are corrected to account for the reduced length to diameter. 2. Cores were received dry. 3. Specimen E4(2) B was used to determine the test load for the Static Chord Modulus of Elasticity test. 4. Remaining cores were tested for compressive strength after Static Chord Modulus of Elasticity testing.

Date reported : 28/03/11

D. Wong

IANZ Approved Signatory
D. Wong
Designation : *Concrete Technologist*
Date : 28/03/11

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CSF 2003 (13/9/2006, modified 07/01/2008) Page 1 of 1

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B. LINE 1 LEVEL 4 SOUTH WALL: E4 STATIC CHORD MODULUS OF ELASTICITY

STATIC CHORD MODULUS OF ELASTICITY OF CONCRETE IN COMPRESSION TEST REPORT

OPUS INTERNATIONAL CONSULTANTS

Hyland Fatigue & Earthquake Engineering Ltd
P.O. Box 97282
Manukau
Auckland 2241

Attenion: Clark Hyland

Project :	**CTV Building**
Location:	**E4: Line 1 Shearwall**
Client :	**Hyland Fatigue & Earthquake Engineering Ltd**
Sampled by :	**Concut**
Date sampled :	**12/03/11**
Sample description :	**Nominal 95mm Ø cores**
Mix identification:	**-**
Date received:	**17/03/11**

Project No :	**6JHFEE.11/6CL**
Lab Ref No :	**4-11/027**
Client Ref No :	**-**

Test Results

Test date:	25/03/11			
Compressive strength:	31.0 MPa			
Compressive stress at test load:	12.5 MPa			
Sample ID:		E4 (1)	E4 (2) A	E4 (3)
Average diameter:	(mm)	95.7	95.7	95.6
Length:	(mm)	192.1	194.1	191.3
Age at test:	(days)	unknown	unknown	unknown
Chord Modulus of Elasticity:	**(MPa)**	**24,000**	**28,000**	**28,000**
Average:	**(MPa)**		**27,000**	

Test Method: **AS 1012.17 - 1997**
Method 1 (Clause 2.5.1 [a]) was used to determine the test load based on the compressive strength of the concrete. Compressive strength was measured on one cylinder prior to measuring elastic modulus.
Samples were conditioned dry for 7 days in a CTRH room at 22+/-1 deg C until tested.
Samples were tested at room temperature of 23 deg C & 48% RH.

D. Wong
Designation : *Concrete Technologist*
Date : 29/03/11

CSF 2005 (18/09/2006)

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C. LINE 5 LEVEL 1 STAIR WALL NORTH CORE: COMPRESSIVE STRENGTH

CONCRETE CORE COMPRESSION TEST REPORT

OPUS INTERNATIONAL CONSULTANTS

Hyland Fatigue & Earthquake Engineering Ltd
P.O. Box 97282
Manukau
Auckland 2241

Attenion: Clark Hyland

Project :	**CTV Building**
Location :	**LW: Lift Core Walls**
Client :	**Hyland Fatigue & Earthquake Engineering Ltd**
Contractor :	-
Sampled by :	**Concut**
Date poured :	-
Sampling method :	**Core**
Grade :	-
Actual slump (mm) :	-
Specified slump (mm) :	-
Date received :	**17/03/11**

Project No :	**6JHFEE.11/6CL**
Lab Ref No :	**4-11/028**
Client Ref No :	-

Test Results

Date tested			28/03/11	
Age at test	(days)		-	
Sample ID		LW(1)	LW(2)	LW(3)
Average diameter	(mm)	92.4	92.6	92.1
Length	(mm)	191.0	192.5	190.0
Length to diameter ratio		2.07	2.08	2.07
Compressive strength	(MPa)	33.0	34.0	39.5
Corrected compressive strength [1]	(MPa)	-	-	-
Ave compressive strength	**(MPa)**		**35.5**	
Ends capped		2	2	2
Defects prior to testing		*Note 3*	*Note 3*	-
Failure mode		Shear	Normal	Normal
Conditioning type [2]		Dry	Dry	Dry

Test Methods	Notes
Compressive strength: NZS 3112:Part 2: 1986 Clause 9	1. Strengths of specimens with length to diameter ratio 1.00 to
Capping: NZS 3112 : 1986, Pt 2 Clause 4 (amendment No 2 2000)	1.90 are corrected to account for the reduced length to diameter.
Correction for length to diameter ratio: in-house method CL-04-511.	2. Cores were received dry.
No corrections made for presence of rebar in core.	3. Horizontal D12 rebar in top 1/3 of specimen.

Date reported : 29/03/11

D. Wong

IANZ Approved Signatory
D. Wong
Designation : *Concrete Technologist*
Date : 29/03/11

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D. LINE 5 LEVEL 1 STAIR WALL IN NORTH CORE: STATIC CHORD MODULUS OF ELASTICITY

STATIC CHORD MODULUS OF ELASTICITY OF CONCRETE IN COMPRESSION TEST REPORT

OPUS INTERNATIONAL CONSULTANTS

Hyland Fatigue & Earthquake Engineering Ltd
P.O. Box 97282
Manukau
Auckland 2241

Attenion: Clark Hyland

Project :	**CTV Building**
Location:	**LW: Lift Core Walls**
Client :	**Hyland Fatigue & Earthquake Engineering Ltd**
Sampled by :	**Concut**
Date sampled :	**15/03/11**
Sample description :	**Nominal 95mm Ø cores**
Mix identification:	**-**
Date received:	**17/03/11**

Project No :	**6JIIFEE.11/6CL**
Lab Ref No :	**4-11/028**
Client Ref No :	**-**

Test Results

Test date: 28/03/11

Average mass/unit volume: 2,340 kg/m^3
Maximum strain to be applied: 450 microstrain

Sample ID:		LW (1)	LW (2)	LW (3)
Average diameter:	(mm)	92.4	92.6	92.1
Length:	(mm)	191.0	192.5	190.0
Density (uncapped)	(kg/m^3)	2,330	2,350	2,330
Age at test:	(days)	unknown	unknown	unknown
Chord Modulus of Elasticity:	**(MPa)**	**29,000** *Note 1*	**28,000** *Note 1*	**29,000**
Average:	**(MPa)**		**29,000**	

Note 1. Horizontal D12 rebar in top 1/3 of core.

Test Method: AS 1012.17 - 1997
Method 3 (Clause 2.5.1 [c]) was used to determine the test load which is based on the mass/unit volume of the concrete.
Samples were conditioned dry in a CTRH room at 22+/-1 deg C until tested.
Samples were tested at room temperature of 23 deg C & 43% RH.
No allowance was made for presence on rebar in cores LW(1) & (2).

D. Wong
Designation : *Concrete Technologist*
Date : 29/03/11

CSF 2005 (13/09/2006)

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E. SUSPENDED FLOOR SLAB CONCRETE CORES (ITEMS E14 AND E23)

CONCRETE COMPRESSION OF CORES TEST REPORT

OPUS

Project :	**Material Strength Investigation**
Location :	**CTV Building, Christchurch**
Client :	**Hyland Fatigue & Earthquake Engineering Limited**
Contractor :	**Concut Limited**
Sampled by :	**Concut Limited (John)**
Date sampled :	**12 March 2011**
Sampling method :	**Concrete Hole Saw (Horizontal)**
Sample description :	**Drilled Concrete Core**
Sample condition :	**Damp as received**
Date cored :	**12 March 2011**
Source of concrete :	**CTV Building, E14 Floor Slab & E23 Hi-bond suspended floor slab**
Grade of concrete :	**Not Advised**
Design strength :	**Not Advised**
Actual slump :	**Not Advised**
Date laid :	**Not Advised**

Project No :	6-HFEE.11/006LC
Lab Ref No :	5674
Client Ref No :	Clark Hyland

Test Results

Lab reference no		056	056	056	056	056	056
Client reference no		E14	E14.2	E14.3	E23.1	E23.2	E23.3
Date tested		28/03/11					
Dry cured	(days)	7					
Size & position of any reinforcement		No Steel	6mm Re-bar	No Steel	6mm Re-bar	6mm Re-bar	6mm Re-bar
Visual description		Horizontal Core					
Average core diameter	(mm)	57.4	57.4	57.5	57.3	57.0	57.3
Average core length	(mm)	117.5	115.5	107.2	116.1	115.5	118.6
Density	(kg/m^3)	2359	2380	2355	2356	2347	2358
Height diameter ratio		2.05	2.01	1.86	2.03	2.03	2.07
Conditioning		Dry					
Load at failure	(kN)	64.9	79.2	69.4	61.4	57.8	49.8
Compressive strength	(MPa)	25.0	30.5	26.5	24.0	22.5	19.5
Type of fracture		Shear	Cone/Split	Cone/Split	Shear	Column	Cone/Split

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

Sampling is not covered by IANZ Accreditation. Results apply only to sample tested.
This report may only be reproduced in full

IANZ Approved Signatory

Designation : *Laboratory Manager*
Date : 29 March 2011

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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F. PRECAST LOG BEAM: COMPRESSIVE STRENGTH

CONCRETE COMPRESSION OF CORES
TEST REPORT

OPUS

Project :	**Material Strength Investigation**
Location :	**CTV Building, Christchurch**
Client :	**Hyland Fatigue & Earthquake Engineering Limited**
Contractor :	**Concut Limited**
Sampled by :	**Concut Limited (John)**
Date sampled :	**12 March 2011**
Sampling method :	**Concrete Hole Saw (Vertically)**
Sample description :	**Drilled Concrete Core**
Sample condition :	**Damp as received**
Date cored :	**12 March 2011**
Source of concrete :	**CTV Building, Logbeam pre-cast (cored vertically)**
Grade of concrete :	**Not Advised**
Design strength :	**Not Advised**
Actual slump :	**Not Advised**
Date laid :	**Not Advised**

Project No :	6-HFEE.11/006LC
Lab Ref No :	5675
Client Ref No :	Clark Hyland

		Test Results
Lab reference no		057
Client reference no		CTV Logbeam
Date tested		28/03/11
Dry cured	(days)	7
Size & position of any reinforcement		10mm Re-bar
Visual description		Vertical Core
Average core diameter	(mm)	107.2
Average core length	(mm)	215.2
Density	(kg/m^3)	**2330**
Height diameter ratio		2.01
Conditioning		Dry
Load at failure	(kN)	225.1
Compressive strength	(MPa)	**25.0**
Type of fracture		Column

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

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

Designation : *Laboratory Manager*
Date : 29 March 2011

laboratory

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

PF-LAB-098 (18/12/2010)

Page 1 of 1

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APPENDIX F: STRUCTURAL AND ARCHITECTURAL DRAWINGS

Portions of structural and architectural drawings prepared by DENG and ARCH are shown to aid with interpretation of the report. Portions are included with the permission of DENG and ARCH.

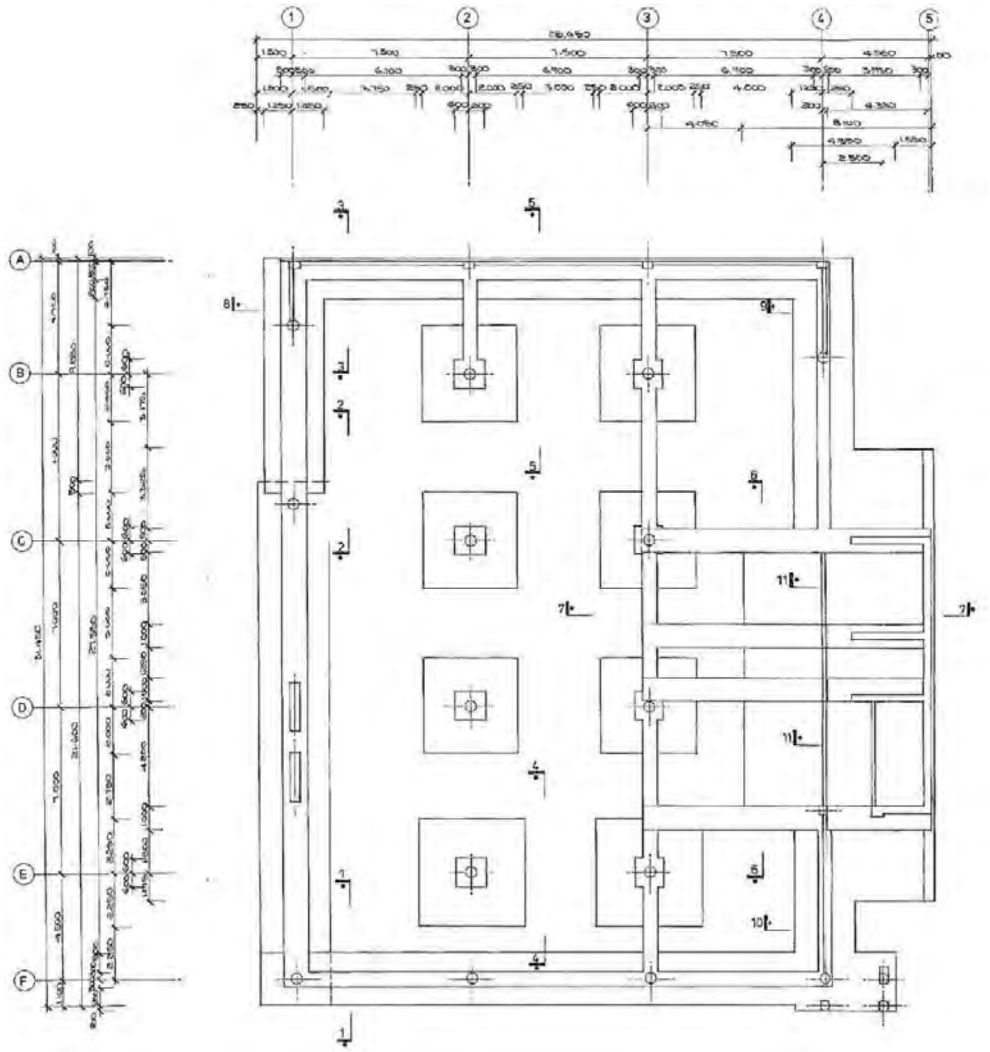

Figure 63 - Foundation Layout (Extract from DENG Dwg S2)

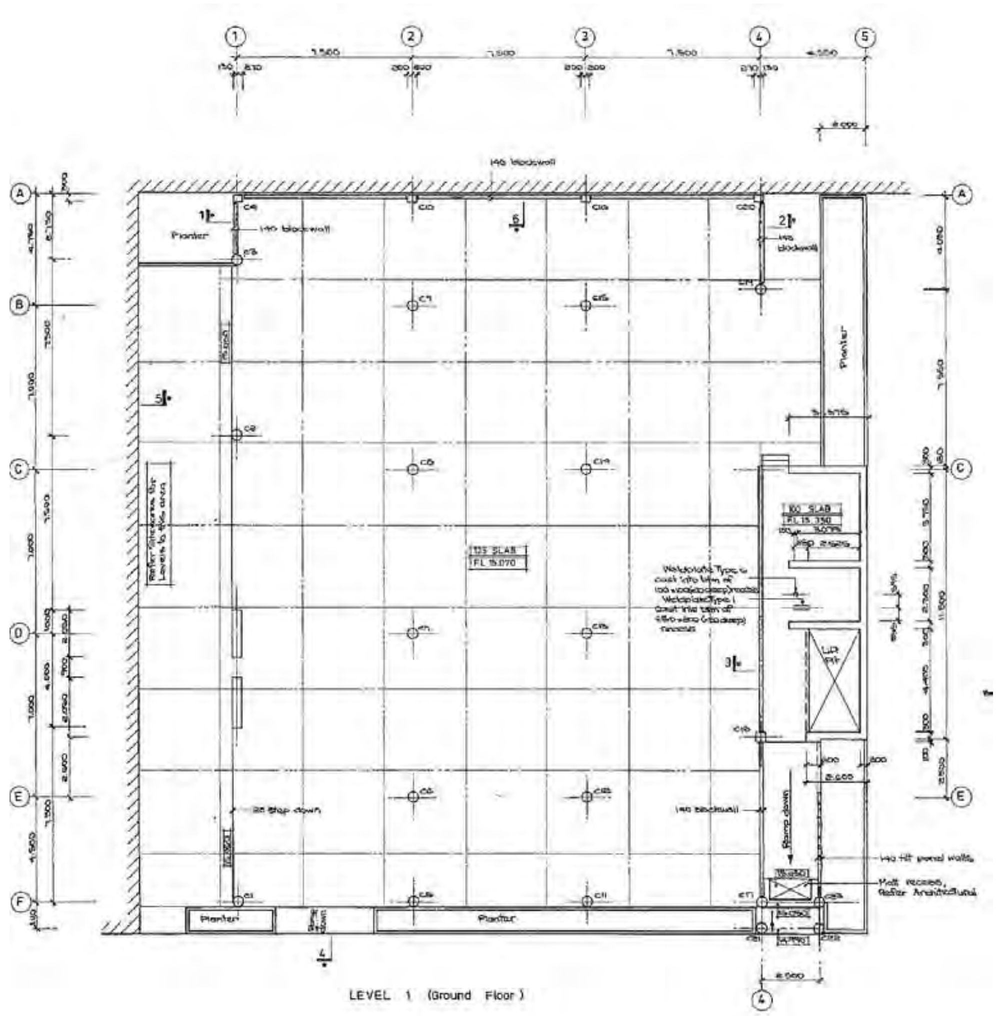

Figure 64 - Level 1 ground floor slab layout (extract DENG Dwg S9)

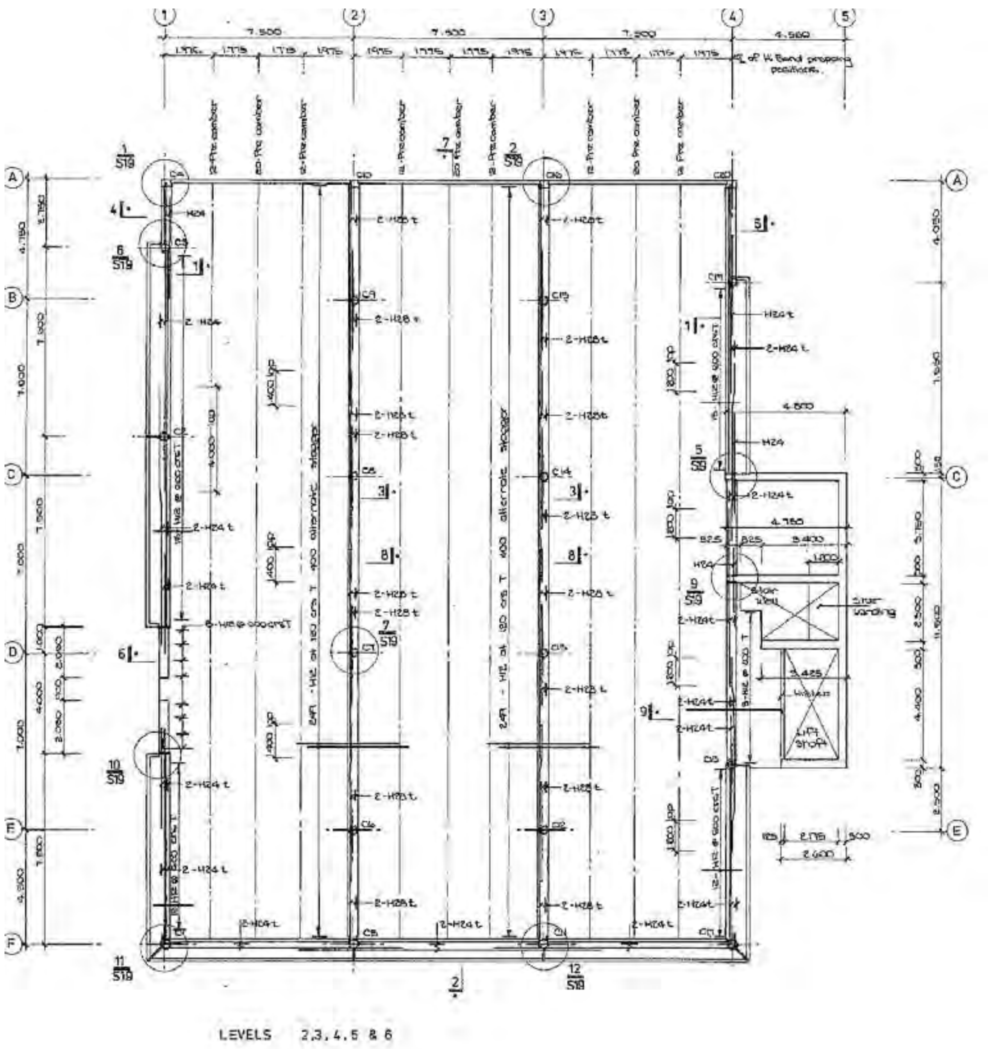

Figure 65 - Level 2 to 6 Floor Layout (Extract from DENG Dwg S15)

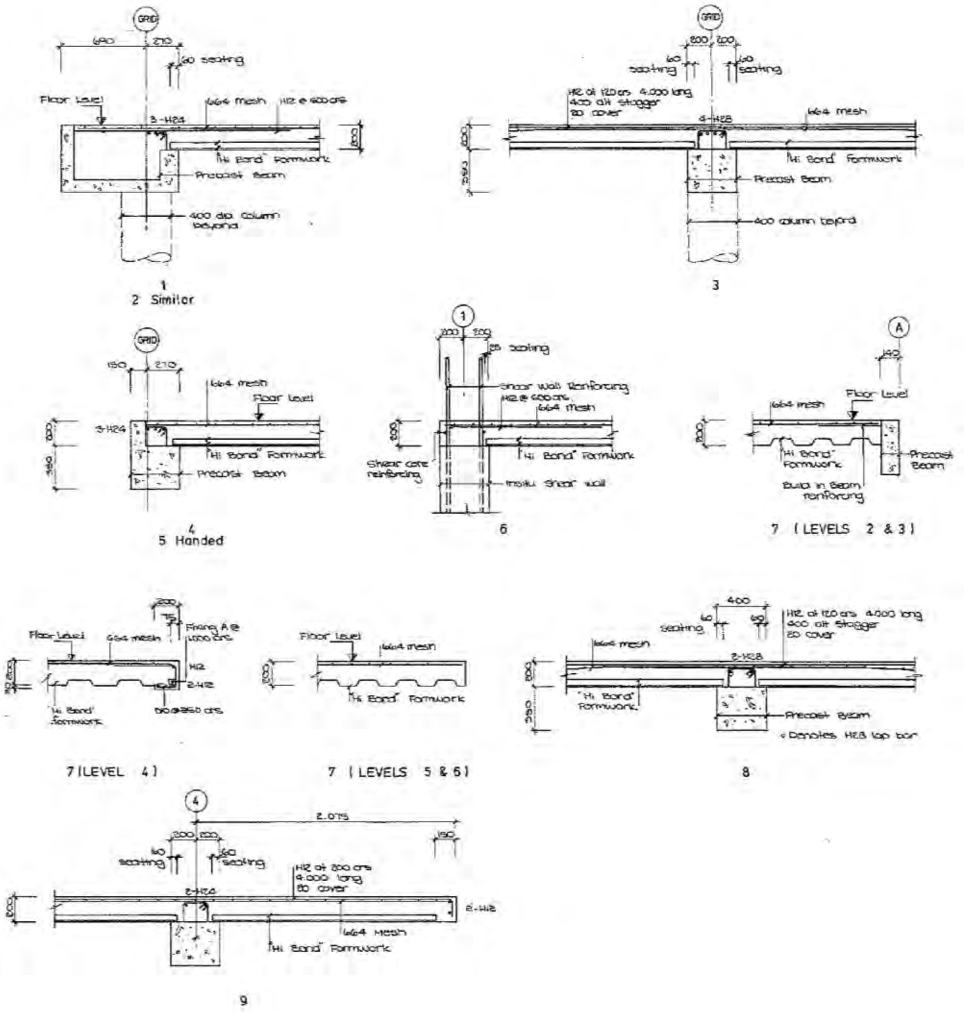

Figure 66 - Level 2 to 6 floor slab details (Extract from DENG Dwg S15)

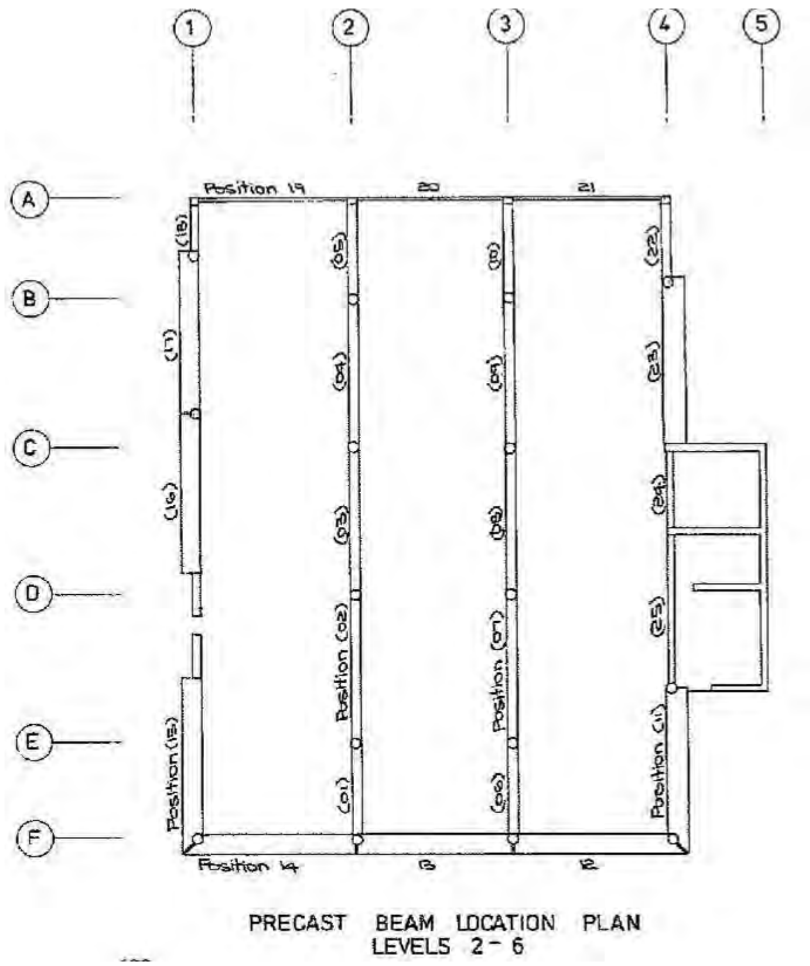

Figure 67 - Precast beam layout drawings (Extract DENG Dwg S18

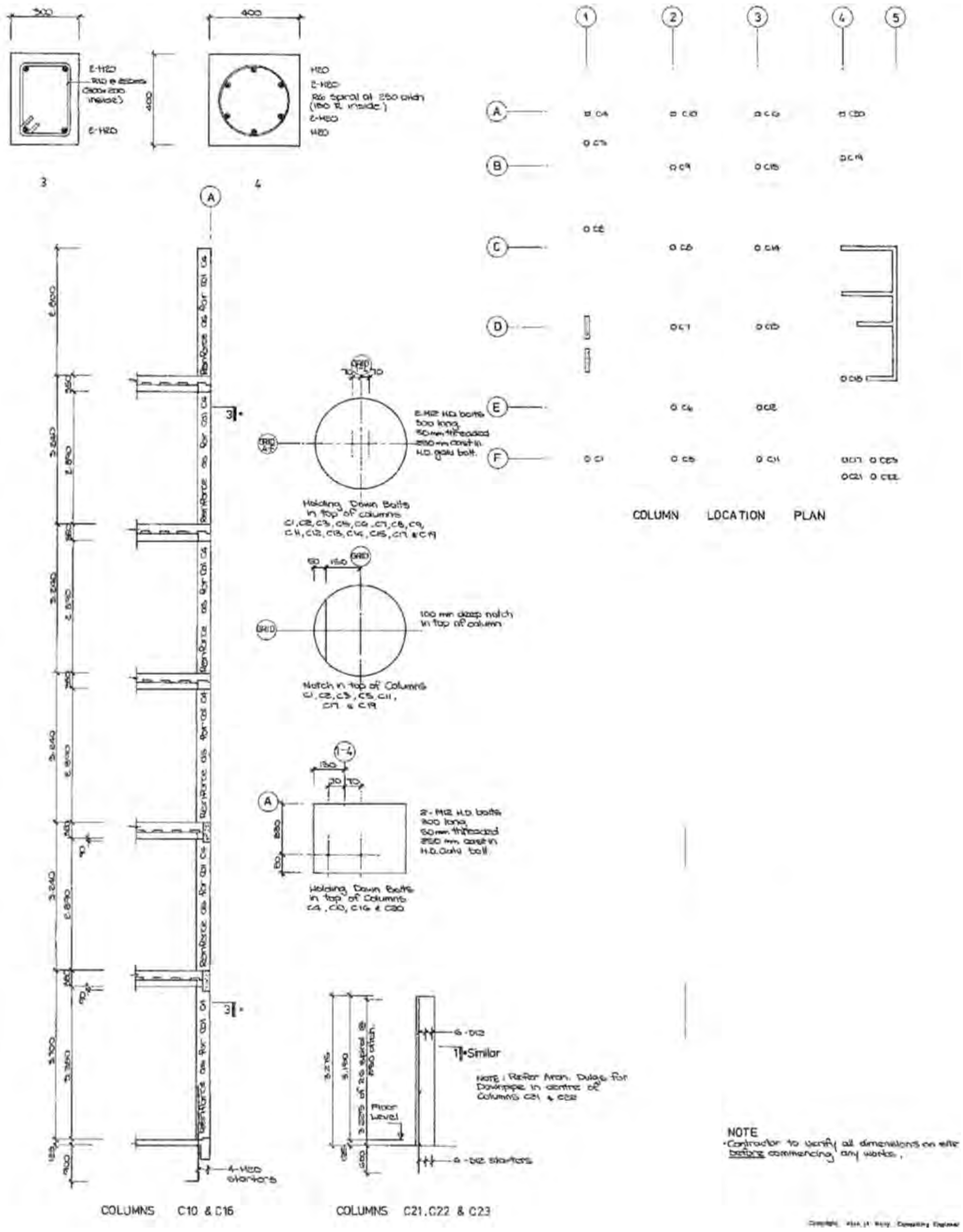

Figure 68 - Columns (Extract DENG Dwg S14)

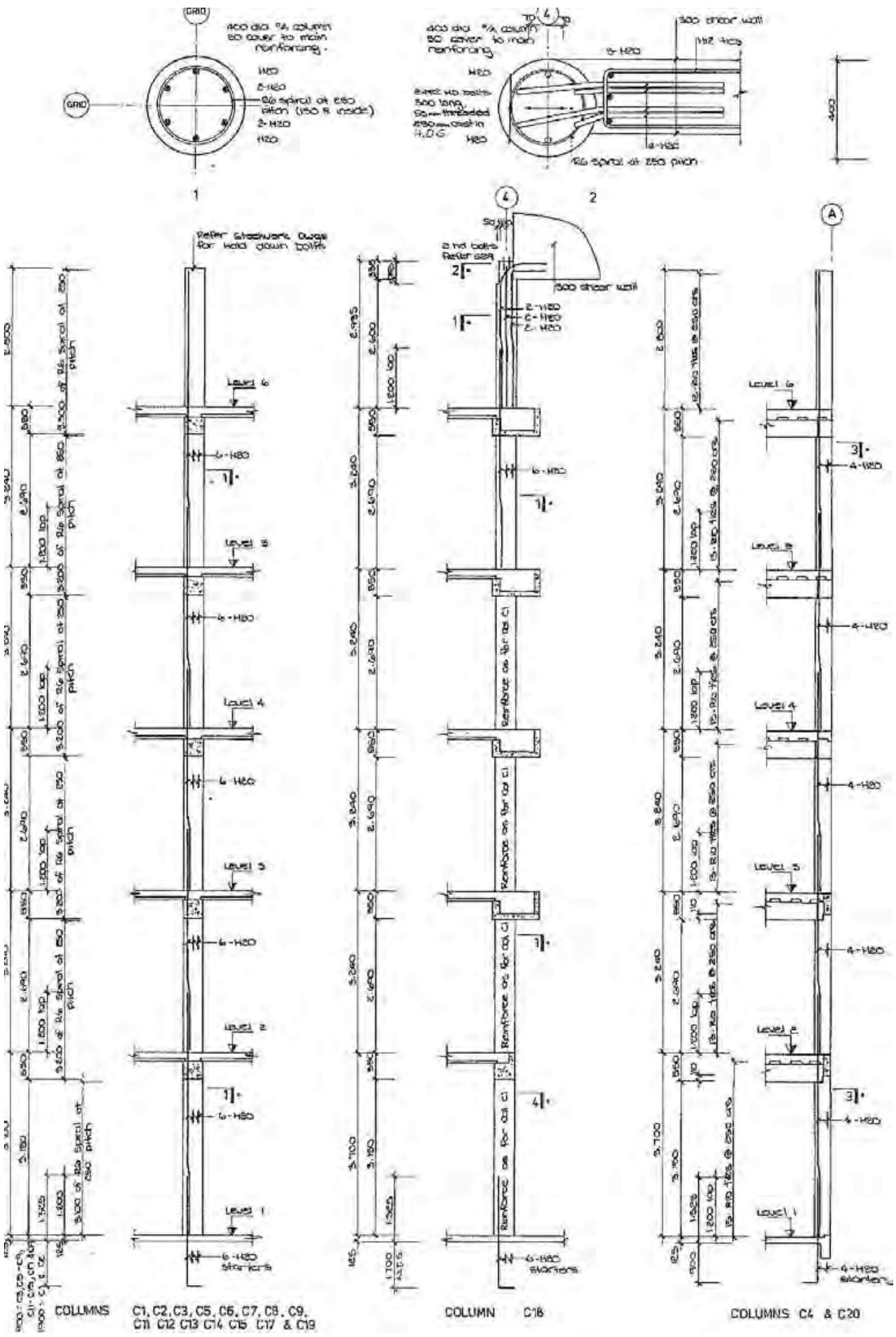

Figure 69 - Columns (Extract DENG Dwg S14)

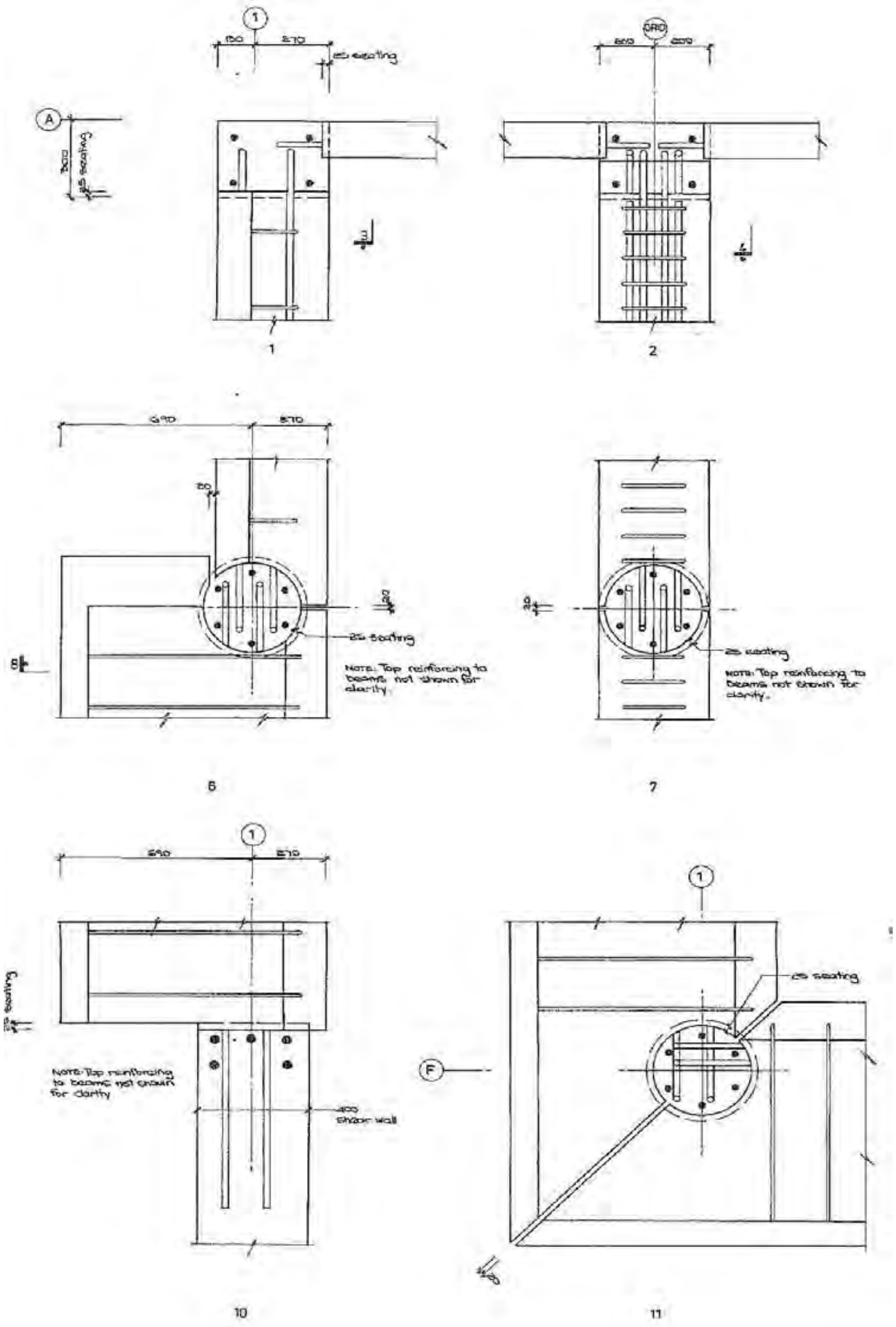

Figure 70 - Beam-Column Joints (Extract DENG Dwg S19)

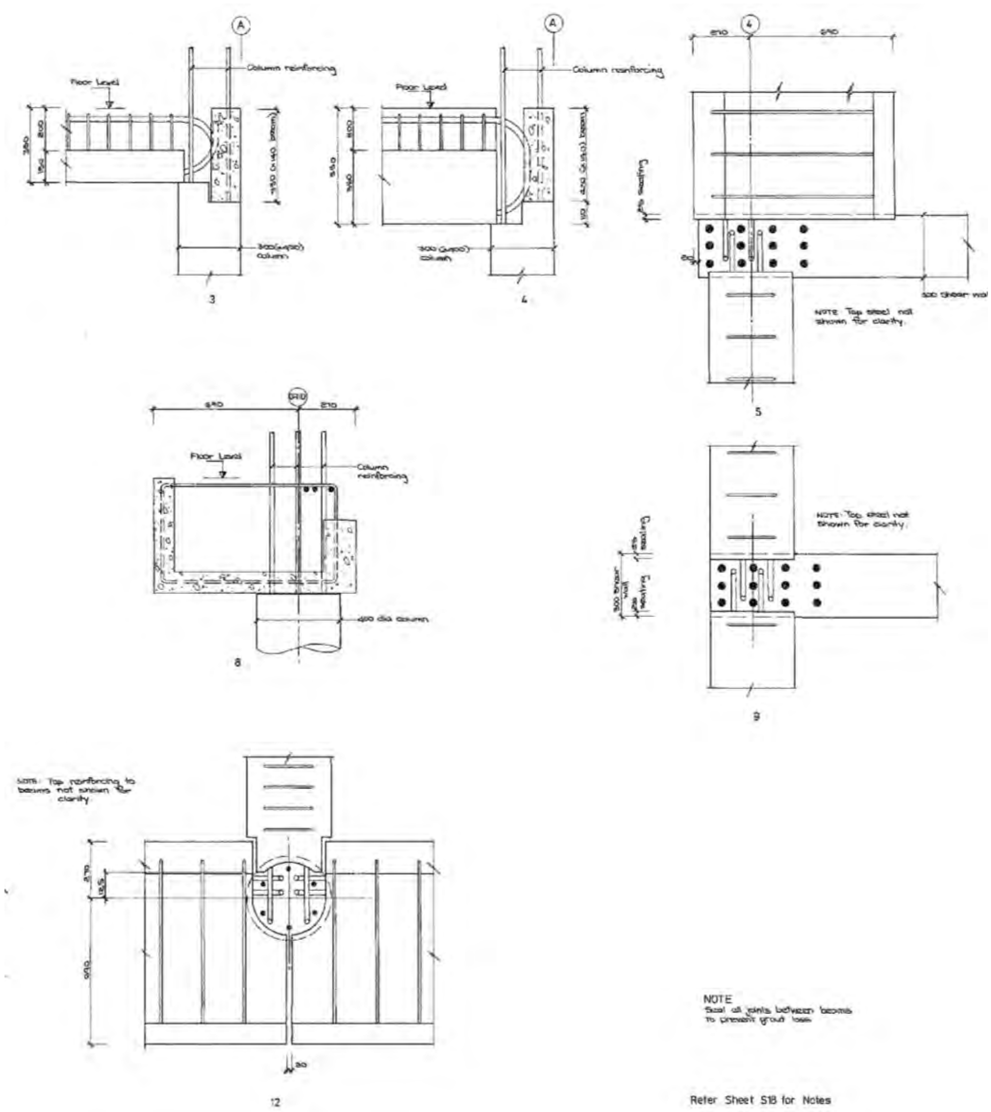

Figure 71 - Beam-Column Joints (Extract DENG Dwg S19)

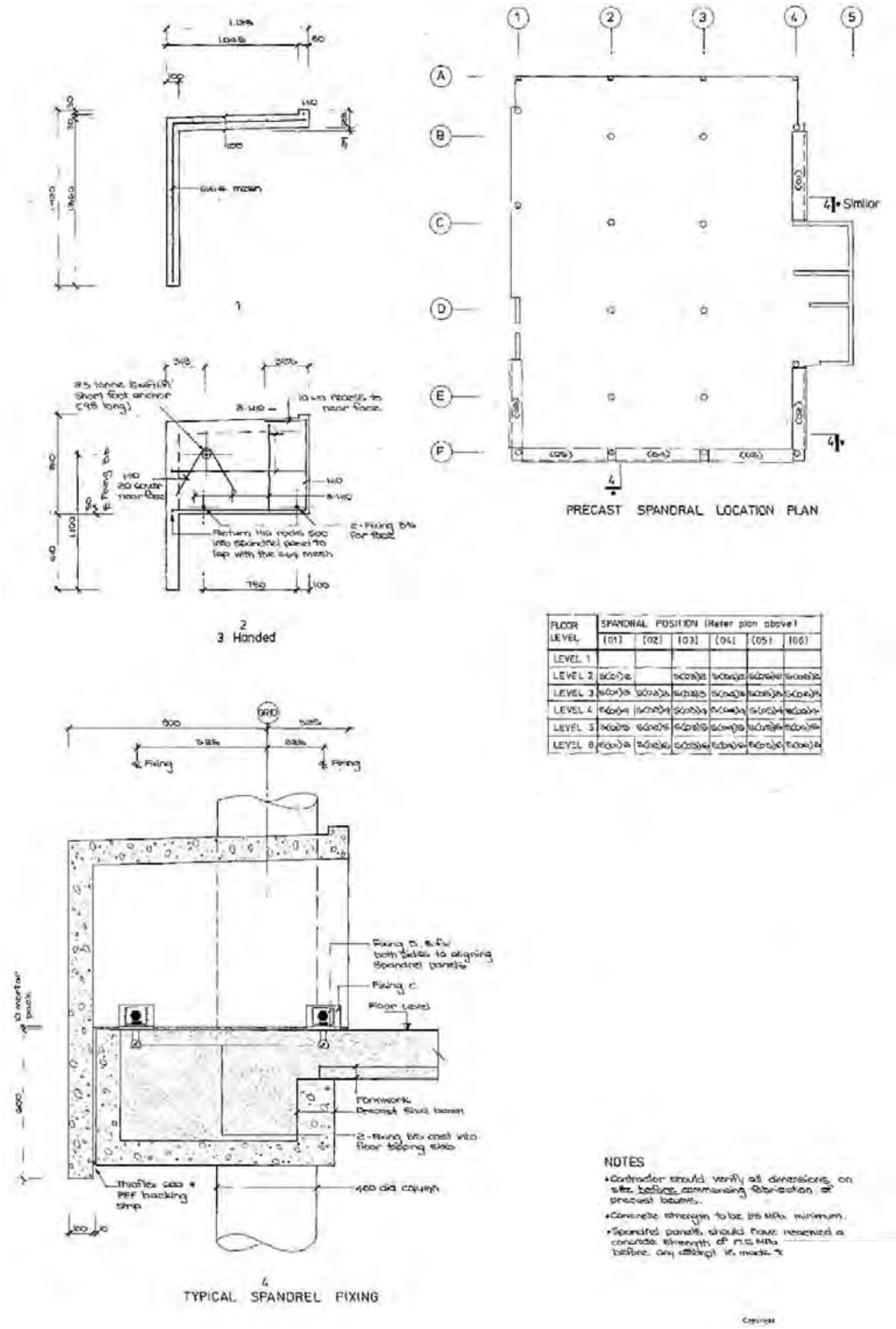

Figure 72 - Pre-cast spandrel panels (Extract from DENG Drawing S25)

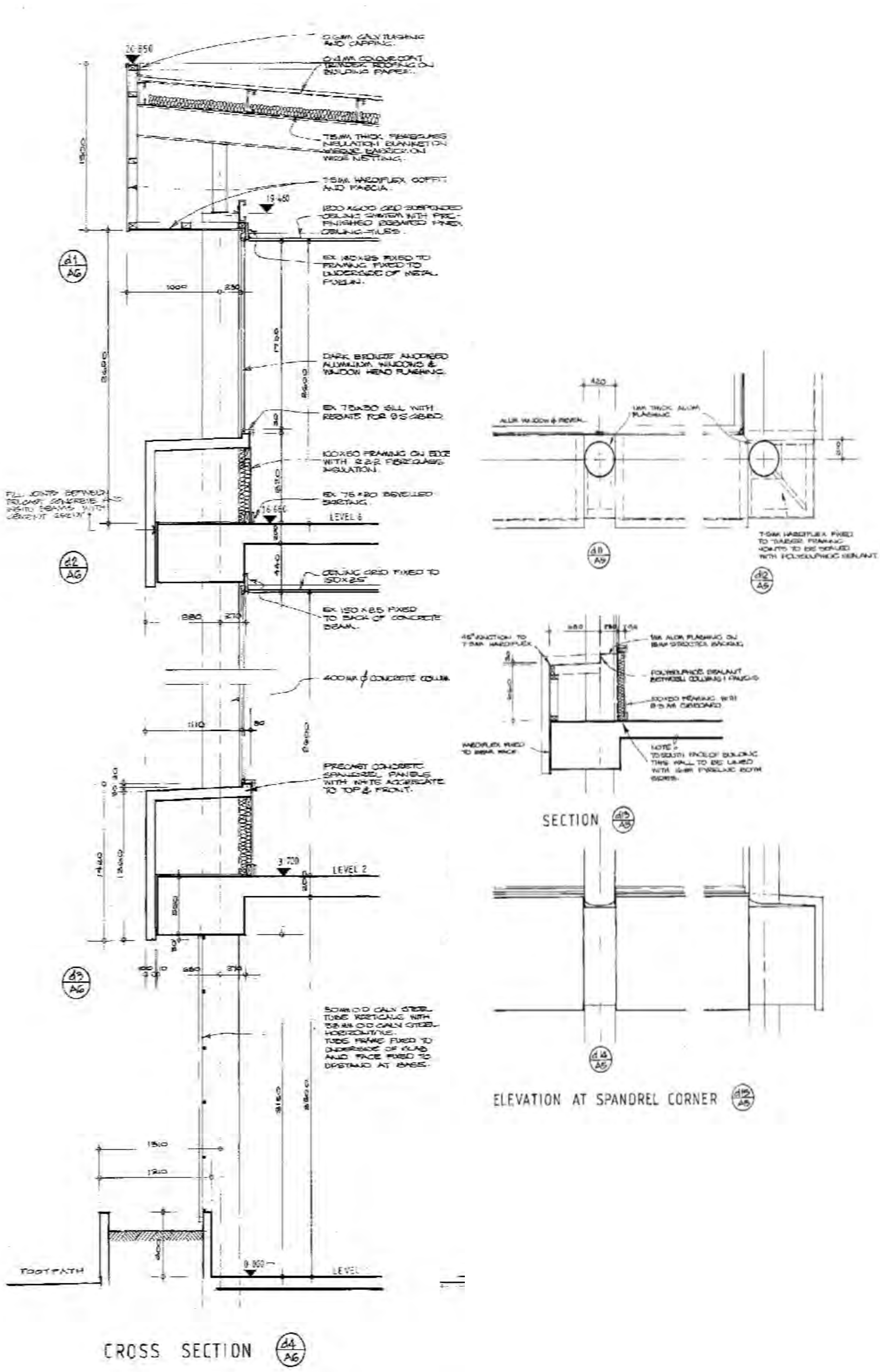

Figure 73 - Spandrel Panel Details at 400 mm Diameter Columns (Extract ARCH Dwg A7)

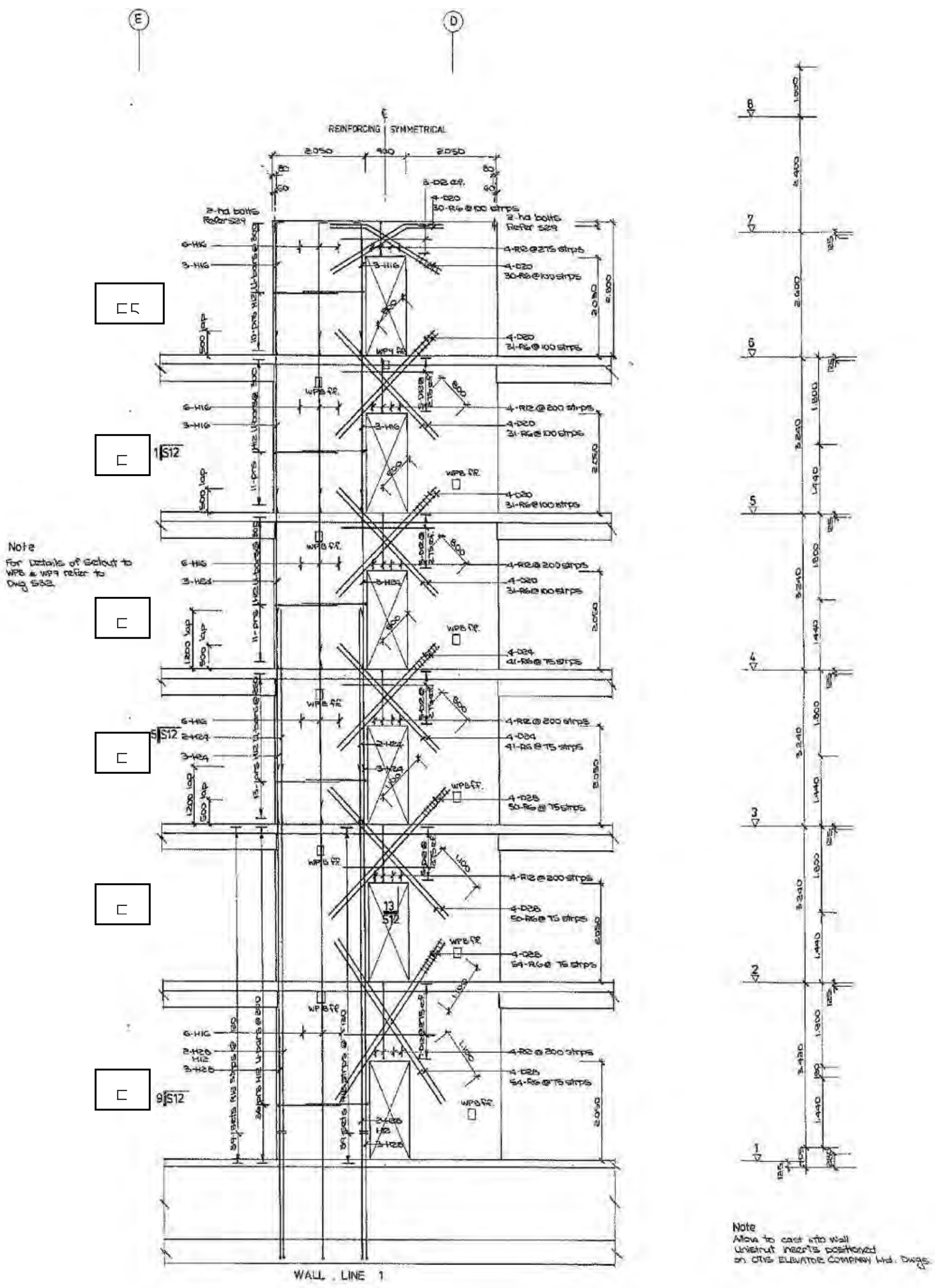

Figure 74 - Line 1 South Wall with Items E1 to E5A identified (Extract from DENG Dwg S10)

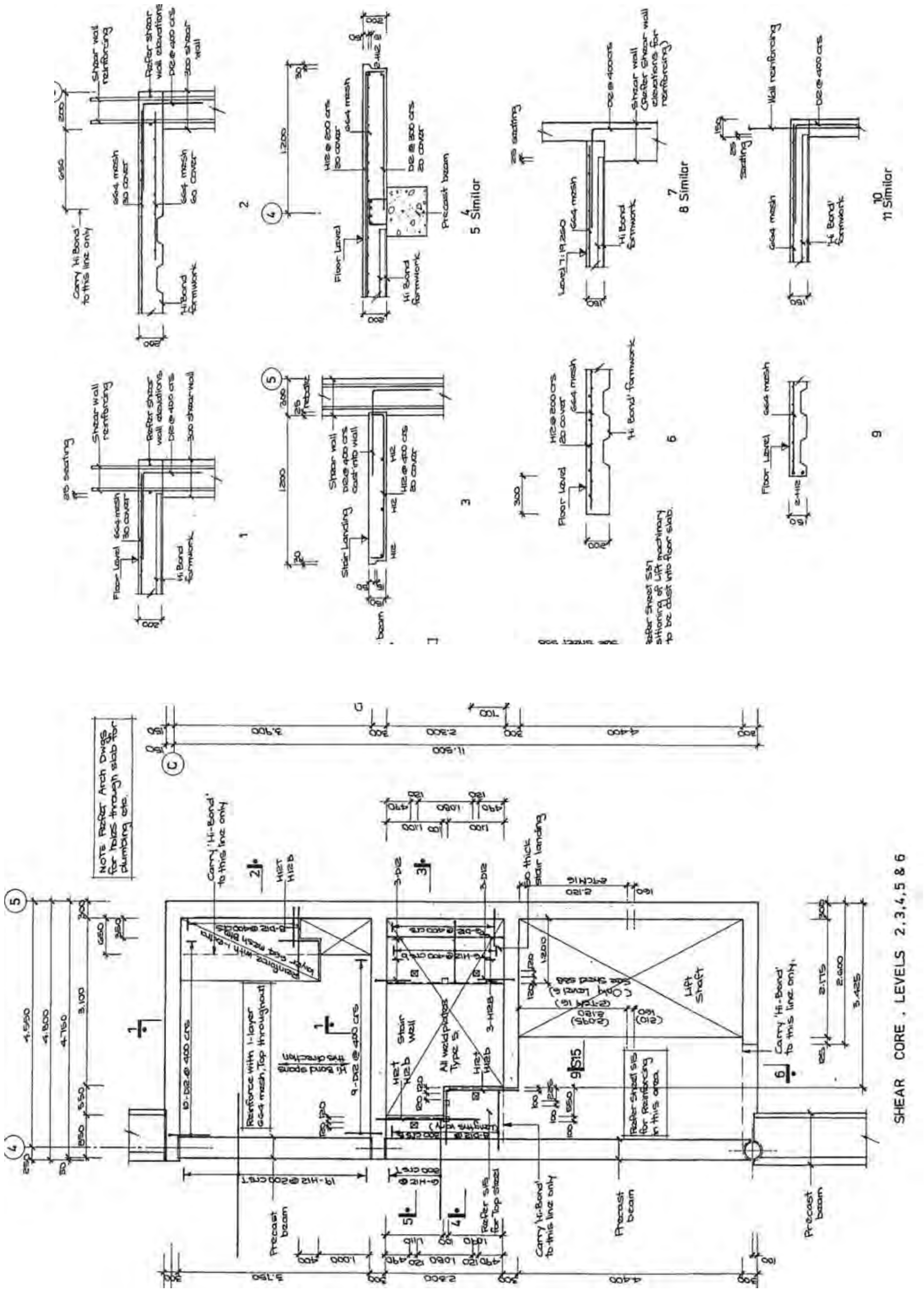

Figure 75 – North Core and slab sections (Extract from DENG Dwg S10)

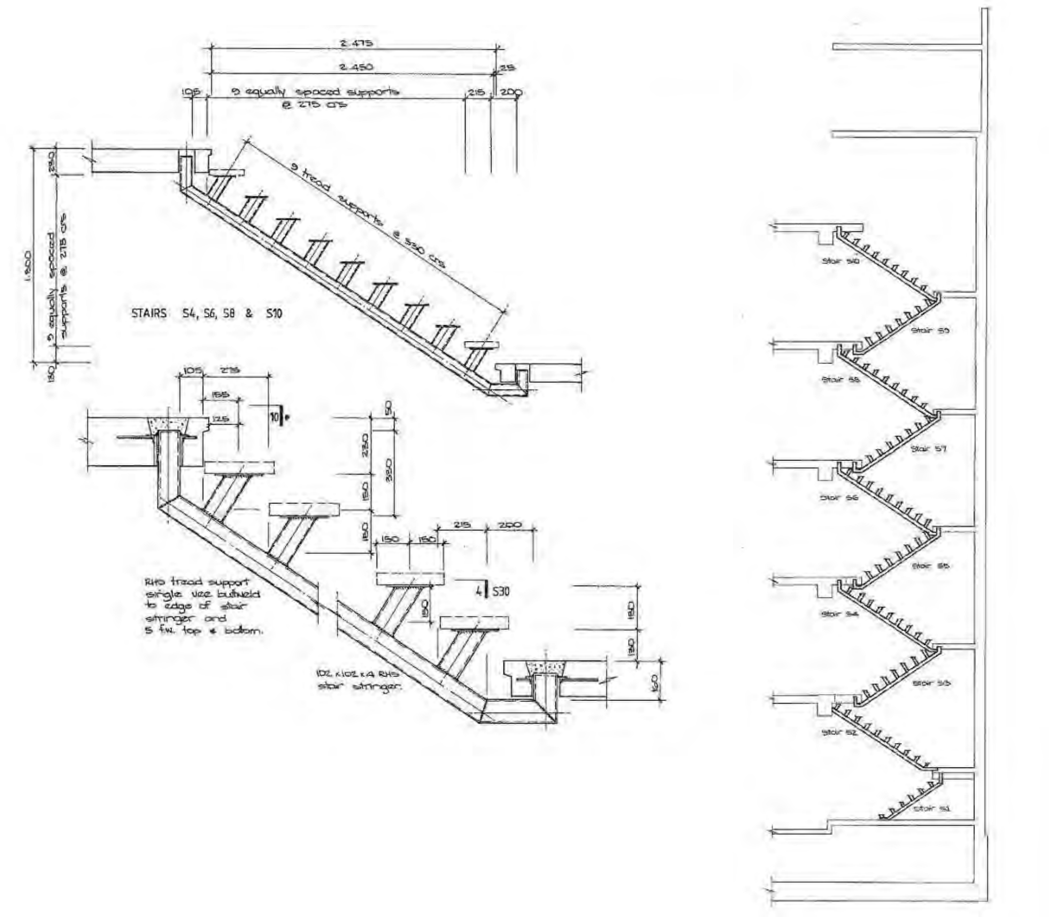

Figure 76 - Line 4 to 5 Stairs and detail of Stair S8 Level 4 to 5 (extract from DENG Dwg S31)

APPENDIX G: STRUCTURAL SPECIFICATION

A. CONCRETE AND REINFORCING STEEL SPECIFICATION

2503

2. CONCRETE & REINFORCING STEELWORK.

2.1 GENERAL
Refer to the General and Special Conditions of Contract Clauses which shall apply to all work in this section of the Specification.

2.2 SCOPE
This section of the specification includes the supply, forming and casting of all cast-in-place, plain and reinforced concrete including all items necessary to complete the work indicated on the drawings and not specifically described elsewhere in this Specification. This section of the Specification includes the supply, erection, reinforcing and casting of the components of the approved proprietary floor system specified in Clause 2.16 of this Specification.
This section of the Specification includes the erection of all precast concrete. The PRECAST CONCRETE section includes manufacture of precast concrete units as detailed and delivery to the site if necessary.

2.3 MATERIALS AND WORKMANSHIP
The Contractor shall comply with all requirements of NZS 3109:1980 except where specified otherwise herein or instructed otherwise by the Engineer. A copy of this standard shall be kept on the site and relevant parts read with the following clauses of the Specification.

2.4 CONCRETE
Site concrete and concrete required to make good excavations shall be 10 MPa at 28 days or better.
All other conrete shall be SPECIAL ro HIGH GRADE, from an approved ready-mix plant, and as defined in NZS 3109: Clause 6.2 and of the following strengths:-

Foundation beams and pads	20 MPa
Columns at Level 1	35 MPa
Columns at Level 2	30 MPa
Columns at Level 3	25 MPa
All other structural concrete including floors and walls	25 MPa

The maximum aggregate size shall be 19mm.

2.5 CONCRETE TESTS
The ready-mix supplier shall make control tests in accordance with NZS 3104, and shall pay the costs of such tests. Tests shall be made either at the ready-mix plant or at the site, except that if the Engineer specifically calls for tests at the site as a result of any dissatisfaction with the plant testing procedure, these shall be done by the ready-mix supplier.

2. cont'd... 2503

2.6 REINFORCEMENT
All reinforcement shall comply with NZS 3402 (1973). Bars prefixed with a 'D' on the drawings shall be deformed Grade 275 steel.
Bars prefixed with a 'R' on the drawings shall be plain Grade 275 steel.
Bars prefixed with an 'H' on the drawings shall be deformed Grade 380 steel.
Mesh shall be hard drawn steel wire fabric to NZS 3422 (1972). All reinforcement and workmanship shall conform to the requirements of NZS 3109:1980.

2.7 FAIRFACE FINISHES
All concrete surfaces that will be visible in the finished job, or covered with paint, Enduit plaster, or tiles, shall be finished fairface.
All concrete required to have a fairface finish shall be cast to a high standard using accurately constructed form work and to a high standard of workmanship. In addition to surface tolerances specified below, the finished surface shall conform for blowholes with illustration 4 in the NZ Standard NZS 3114:1980 "Specification for Concrete Surface Finishes." Refer to the Architect's drawings for the finish required on concrete surfaces.

2.8 SLAB FINISH
Except as specified below, all slabs have a steel trowelled finish. Screed off and lightly wood float. Finish slabs with approved power floating and compacting machines to leave a dense, level surface which does not vary more than 6mm from a 3 metre straight edge, and not more than ± 15mm from true level.

2.9 SITE CONCRETE
Form and cast 50mm site concrete beneath main foundations and elsewhere as necessary to provide a clean, dry working platform. Ensure ground surface is clean and dry and there is no evidence of soft spots.

2.10 FOUNDATIONS
Form and cast main foundation beams as detailed. It is envisaged that the beams will be cast in stages with construction joints.
Allow to scrabble or green cut the faces of these joints. The exact location and details of all construction joints are to be agreed with the Engineer before pouring concrete.

2.11 LIFT PIT
Form and cast lift pit walls and floor with sump as detailed. Build in PVC 140mm HYDROFOIL waterstop or similar to all construction joints in floor and walls. Waterproof the concrete with SIKA Plastocrete-N-Waterproofer or approved equivalent.

2 cont'd... 2503

2.12 GROUND FLOOR SLAB
Form and cast ground floor slab on damp proof course on compacted hardfill. Cast in strips and sawcut into panels where agreed by the Engineer on site. The maximum spacing of sawcuts or construction joints shall not exceed 3.75 metres.

2.13 PROPPING OF PRECAST BEAMS
Precast beams shall be propped to support the dead weight of the beam until the floor concrete has reached 20 MPa.

2.14 CHASES, HOLES AND NIBS
Form all chases, holes, upstands and nibs as shown on the drawings or required by other trades. Chases and holes shall be accurately positioned and formed at the time of casting the concrete.
Set concrete shall not be hacked unless specific approval is obtained from the Engineer.

2.15 BUILDING IN
As the work proceeds, build in all necessary bolts and other fixings. The Concretor shall ascertain from all other sub-contractors all particulars relating to their work with regard to order of its execution and details of all such provisions of fixings sleeves, chases, holes, etc., and of all necessary items to be built into concrete and shall ensure that all such items are provided for and/or positioned.

No claim will be recognized or allowed for at extra cost of cutting away or drilling concrete work already executed in consequence or any neglect of the Contractor to ascertain these particulars and make the necessary provision beforehand.

2.16 FLOOR SLABS
Concrete floors have been detailed to use the 'DIMOND HI-BOND H.S.' composite steel/concrete floor system. This has a profiled metal deck of 54mm overall depth, made from G500 steel, 0.75mm thick.

The floor shall be handled, laid, and fixed in accordance with the manufacturer's written "laying instructions".
Provide temporary propping to floors as shown on the drawings, with an upward camber to the propping lines as detailed. Floors shall be constructed of a uniform thickness, so that slab surfaces as constructed shall follow the cambered profile of the floor decking.
Propping shall extend over at least three levels at all times, to distribute the weight of the floor being poured into three lower floors, and to support mobile scaffolds being used to erect precast floor beams.

2503

3. PRECAST CONCRETE

3.1 GENERAL

Refer to the General and Special Conditions of Contract clauses which shall apply to all work in this section of the Specification.

3.2 SCOPE

This section of the specification includes the manufacture and supply on site of the following pre-cast units:-

1. Precast beams
2. Precast wall panels

The work includes the fabrication and supply of all structural steel fittings to be built into the units as detailed on the drawings.

3.3 MATERIALS AND WORKMANSHIP

All formwork, concrete and concreting and finishing shall be in accordance with the relevant clauses of Concrete and Reinforcing Steelwork Specification except where noted otherwise in this section.

3.4 CONCRETE

All concrete shall be HIGH or SPECIAL GRADE complying with NZS 3109 Clause 6.2.Concrete for all precast work shall be 25 MPa at 28 days with 18mm maximum size aggregate.

3.5 TOLERANCES

All precast units shall be manufactured to the following tolerances unless stated otherwise on the drawings:

- Length	± 6 mm
- Cross Section	± 3 mm
- Squareness (of cross section and ends)	± 3 mm
- Twist (dimensions from plane containing the other three corners	± 3 mm
- Built in Items	± 5 mm

The above tolerances are given as a guide. Their application in any particular case shall be subject to interpretation by the Engineer.

3.6 FINISHES

All precast concrete exposed in the finished building shall be cast to a high standard using accurately constructed formwork and a high standard of workmanship. Precast items that do not meet the required standard to the satisfaction of the Engineer will be rejected. Formwork shall be such as to produce a high quality fair face finish on all exposed surfaces. Formwork shall be made from sheet steel or dressed plywood treated with a polyurethane finish to a high quality smooth surface, or similar.

3. cont'd... 2503

In general finished surfaces shall be smooth and formed with moulds or by careful trowelling. Surfaces shall be free from honeycombing, grout loss, excessive air holes or other imperfections. Arrises shall be straight clean and sharp and free from spalling or damage.
All exposed surfaces shall have a similar appearance and standard of finish. Surfaces finished by trowelling shall be finished to the same standard and uniformly match surfaces against formwork.
Formwork shall be sealed at all corners, joins and inserts to prevent all grout loss.
All surfaces against which concrete is later to be cast shall be left roughened by brooming the poured face while the concrete is still plastic. Clean surfaces thoroughly from all laitance and loose concrete.

3.7 HANDLING
A high standard of finish is required and handling shall be such as to prevent any damage to units.
Approved lifting devices or hooks shall be provided in all precast units and these shall be made available to the Contractor for erection purposes and removed cleanly after use. Units shall be handled only by the hooks or devices provided. They shall be loaded and transported so that no forces are applied in excess of those occurring during normal lifting. Twisting forces shall not be permitted to occur. Units shall be strapped and secured to prevent movement or damage during transportation.

Details of lifting hooks and devices, and their positions, shall be submitted to the Engineer for approval before manufacture commences. Care shall be exercised at all times, that hooks or devices suffer no bending or other damage. Lifting hooks or devices set permanently in the units shall have a safety factor of at least 4 and for repetitive use shall have a safety factor of at least 6.

3.8 STACKING
Units shall be stacked on timber dunnage and suitable soft packing placed under the lifting points. Stacking shall at all times be such as to minimise the effects of creep and to avoid undue distortion of units.
Stacking of units shall be carried out on an area capable of withstanding the bearing pressures involved and in such a way that damage to units, lifting hooks, and to other embedded fixtures and to other units shall not occur.

3.9 MARKING
Mark all units with a mark number, orientation in finished job, and date of casting. The marking shall not be permitted to affect the fairface finish.

3.10 INSPECTION
The Engineer or his representative will inspect the precast units at all stages of manufacture to ensure conformity with this specification. Units which do not conform to the required tolerances, which shown grout leakage, which have been damaged, or which are otherwise defective shall be liable to rejection and may be used in the structure only at the Engineer's discretion.

3. cont'd... 2503

No repair work shall be done without specific instruction from the Engineer.

3.11 BUILDING IN

Supply and fix all lifting bolts, cast in sockets, timber grounds and other fixings as shown on the drawings or as required for the proper erection of the units in the finished work.

3.12 PRECAST SHELL BEAMS

Form and cast the beams as detailed including all reinforcing starters, structural steel fixings, holes for services, rebates, etc, as detailed.
The beams have been detailed to minimise their weight and hence crane capacity. The surface of the beams inside the stirrups shall be roughened to ensure good bond to the infill concrete. Outside the stirrups the surface shall be straight and level to receive the proprietary floor system.

Sides and soffits shall be finished as clause 3.6 where exposed in the completed building, otherwise to a reasonable fairface finish.

Figure 77 - Extract from DENG Pre-cast Concrete Specification