CTV Building: Site Examination and Materials Tests

For: Department of Building and Housing

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1. Executive Summary

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

During the rescue and recovery operation the building was largely deconstructed leaving a pile of debris on the site. Structural remnants were recovered from the debris for examination on 12th 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 it collapsed.

The remnants examined included reinforced concrete columns, the collapsed line 1 shear wall, the line 4 and 5 lift and stair well walls by crane, and various beam and slab items.

The observations made in this report cover only a sample of structural remnants able to be accessed on the site and in the broken up debris deposited at the Burwood Landfill, at the time.

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 earthquake, as well as photos of the structure immediately after the earthquake and during its subsequent de-construction.

A summary of defects that may be relevant to the performance of the structure during the 22 February after-shock are as follows:

1. Concrete strengths were found to be lower than what would have been expected for concrete that had originally complied with the specification during construction.

2. The reinforcing steel was found to have properties consistent with the standards of the time.

3. A portion of reinforcing steel removed from the Line 1 shear wall near ground level was found to have work hardened during the earthquake and prior to the collapse of the building.

4. 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.

5. A northward lean on the Line 4 and 5 lift and stairwell core was found that was concluded not to have been caused by the earthquake and may have occurred during construction.

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

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

8. Connection of the slabs by reinforcing steel into the Line 4 lift core walls was non-existence in some cases at Level 2, 3 and 4.

9. The connection of the C18 column into the lift core wall at Level 7 was less than specified and the bars had de-bonded.

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

11. Rectangular columns which were all located on Line 1 in the structure, typically exhibited beam-column joint failure as well as other damage.

2. 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 causes of damage that led to the collapse of building during the earthquake on 22nd February, 2011.

b. Scope

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

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

c. Background

The CTV Building was located at 249 Madras Street. It was a reinforced concrete building with five suspended floor levels constructed with cast in-situ composite metal deck and concrete floor slabs, precast concrete beams, circular concrete columns, and two sets of shear walls to laterally brace it.

One set of coupled shear walls was located on the Cashell Street or south end (Line 1) to which an external fire escape stair was attached . The other set of shear walls was located at the northern end (Line 4 to 5) and was built around the lift and stair wells.

The development gained a building permit on 30th September, 1986 according to documentation at the Christchurch City Council. Construction was started in 1986 and finished in 1987 or 1988.

The building was severely damaged in the earthquake after-shock on 22nd 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 USAR teams as they searched for and recovered victims from the ruins. Items considered to be of structural significance were marked and set aside by USAR in a pile near the Cashell 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”.

3. Summary of Observations and Findings

a. Composite Metal Deck and Concrete Suspended Slab

The Hi-bond deck that formed the 200 mm thick slab had de-bonded from the underside of the concrete in all cases.

The steel decking had pulled away from the supporting beams in all cases except at the pre-cast beam support on Line 4 at the lift core. In that case the steel decking fractured in tension.

A portion of the decking was tensile tested and found to exceed the specified yield stress of 550 MPa (p.19).

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.

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 (p. 21).

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 beam s 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 5).

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 8).

The lap joint in the exterior columns was concealed by the external spandrel panels and interior linings (Figure 65 and Figure 66).

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 Hi-bond slab had smooth formed unroughened ends at the interface with the beam –column joint zone. This would have reduced beam-column joint shear capacity (Figure 10).

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 unroughened 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 11).

f. Line 1 Shear Wall

The Line 1 shear wall that extended from Level 1 on the ground to the roof had been broken up into single story components during de-construction (Figure 67).

i. Level 1 to 2 (Item E1)
This panel showed flexural cracking patterns typical of cantilever shear walls (Figure 12).

Reinforcing steel taken from eth east end of the wall was found to have yielded and elongated prior to eth collapse of the building (p.59).

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 13).

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 14).

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 57).

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 59).

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 16).

Top surface of wall had smooth rather than roughened construction joint for slab seating.

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

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

vi. Level 6 to Roof (Item E5A)
No obvious cracking in the wall piers or door head coupling beam (Figure 17).

g. Lift and Stairwell Core Walls Line 4 to 5

Horizontal flexural cracking on west and north face at Grid C/5 (Figure 18).

Fine two-way diagonal cracking on the inside faces of Level 1 to 2 walls (Figure 19).

h. Slab and Beam Remnants on Line 4 of Lift and Stairwell Core

The extent of the slabs at the time of examination was measured (Figure 25).

Portions of the level 6 and Level 5 slabs that were still attached immediately after the earthquake 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 20).

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

The Hi-bond steel decking had fractured in tension adjacent to the edge of the fractured slab edge.

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 21).

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 5) was visible in the cover concrete of the wall (Figure 22).

The Hi-bond 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 only and beam-column joint type diagonal cracking was seen on the end of the wall consistent with cyclic demands having occurred during the earthquake.

i. Slab Diaphragm Connections to Lift Core Wing Walls on Grid D and D.5

Drag bar items had been bolted through the slab and into the shear walls at Levels 4, 5 and 6 after the original construction had been completed (Figure 26).

i. Level 2 Connection of Slab to Walls

No reinforcing steel connected the slab to the east wing wall D.5.

A 20mm hole was found in the west wing wall D where a reinforcing bar has pulled out (Figure 27).

ii. Level 3 Connection of Slab to Walls

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.5 to the slab (Figure 28).

iii. Level 4 Connection of Slab to Walls

The drag bar items on both the west and east wing walls had partially fractured in bending and tension. The bolts that passed vertically through the slab and into the drag bar on the west wall had fractured in tension as the slab pried it off as it rotated downwards during the collapse (Figure 29 and Figure 30).

iv. Level 5 and 6 Connection of Slab to Walls

Similar to what was seen at Level 4 (Figure 31and Figure 32).

j. Connection of Column C18 to Lift Core at Level 7

The column had pulled away in tension from the connection at the lift core wall D.5. 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 33).

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 induce by the earthquake could therefore be inferred.

The stair and lift core walls on Line 5 had a northward lean of 91 mm over 18.53 m from Level 1 to Level 7 on the east end and 68 mm on the west end. This is greater than the construction tolerance required in the concrete construction standard NZS3109:1980 of 25 mm.

As no damage was found to the foundation beams around the core and no evidence of settlement could be inferred from the level survey of the slab, overlay and foundation beams it is concluded that the northward lean was caused during the construction of the core walls (Figure 50).

l. Reinforcing Steel Properties

Reinforcing steel samples were extracted from the Line 1 shear wall and tested to determine tensile properties, production uniformity and work hardening during the earthquake.

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 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 showed that the bar had work-hardened during the earthquake 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 39). The chord modulus of elasticity was also determined for the shear wall concrete.

When allowance is made for the expected 25% gain in strength in the concrete over the 25 years since it was poured most of the concrete tested would not have conformed within acceptable confidence limits to the specified 28 day strengths at the time of construction (Table 3).

i. Suspended Slab Concrete Properties

The suspended slab concrete cores achieved average strength at test of 24.7 MPa.

Accounting for strength-aging of 25% the concrete would not have complied with the requirements for the specified 28 day strength of 25 MPa.

It is also would not have complied with the requirements for concrete with 28 day strength of 17.5 MPa with acceptable levels of confidence (p. 66).

ii. Shear wall Concrete Properties

The line 1 and 5 shear wall concrete cores achieved average strength at test of 33.5 MPa.

Accounting for strength-aging of 25% the concrete would not have complied with the requirements for the specified 28 day strength of 25 MPa.

However it would have complied with the requirements for concrete with 28 day strength of 20 MPa with an acceptable level of confidence(p. 67).

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 (p.67).

iii. Column Concrete Properties Summary

In summary at the time of collapse the columns from Level 3 and above are considered to have had properties with the distribution of concrete with specified 28 day strength of 17.5 MPa aged by 25%.

Elsewhere it is considered that some of the concrete columns at Level 1 and 2 had properties consistent with the DENG Specification of 35 and 30 MPa at 28 days respectively and aged by 25%, and some consistent with 17.5 MPa strength at 28 days and aged by 25%.

iv. Level 6 to Roof 400 mm Diameter Column Concrete Properties

The column E25 concrete cores achieved average compressive strength at test of 23.3 MPa.

Accounting for strength-aging of 25% the concrete would not have complied with the requirements for the specified 28 day strength of 25 MPa.

It is also unlikely that it would have complied with the requirements for concrete with 28 day strength of 17.5 MPa (p. 69).

v. Level 1 to 2 400 mm Square Column C18 Concrete Properties

The column C18 concrete cores achieved average compressive strength at test of 16.0 MPa.

Accounting for strength-aging of 25% it would not have complied with the requirements for concrete with 28 day strength of 17.5 MPa nor the 35 MPa strength specified (p. 69).

Silt was found in the concrete cores tested indicating the aggregate and sands had not been adequately washed before being used in the concrete (p.128).

This column was in an area affected by the post-collapse fire. Care was taken to ensure cores were taken away from the surfaces affected by the fire. The samples have been retained for further chemical analysis if required to check for heat effects (Figure 40).

vi. Concrete Properties of 25 Column Remnants from Burwood Landfill

Twenty five column remnants were extracted randomly from the designated CTV debris site at the Burwood Landfill and were tested using rebound hammer techniques and core testing in accordance with ASTM C805 to gain a larger sample of concrete compressive strength properties for the CTV columns (Figure 42).

Seven of the columns were identified as from Level 5 to the Roof, two were from the Level 1 entry way at the northeast corner, and sixteen were of unknown location.

Statistical analysis of the Level 5 to Roof columns identified showed that they would not have complied with the specified 25 MPa strength at 28 days when allowance is made for strength-ageing of 25%. However they could have complied with the requirements for 17.5 MPa strength at 28 days, with an acceptable level of confidence.

Three of the columns from unknown locations had concrete strengths significantly higher than the others. One of these was the lower of two rectangular columns still connected by reinforcing steel.

Their strengths were consistent with a specified 28 day strength of 30 MPa. Which was the strength specified for columns at Level 2.

All the other columns tested of unknown location would not have complied with the minimum specified 25 MPa strength at 28 days, for columns above Level 3, when allowance is made for strength-ageing of 25%.

However they could have complied with requirements for 17.5 MPa strength at 28 days, with an acceptable level of confidence.

4. Examination of Structural Remnants

The examination of structural remnants was undertaken by the author and DBH Structural Engineer, Graeme Lawrence on Saturday 12th March, 2011 (Figure 1). It was then visited again with Ashley Smith of Structuresmith Ltd on 5th April, 2011.

A crane and personnel were provided by John Jones Steel Ltd to move the items around for examination.

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

Figure 1 USAR 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 2).

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 settlement and lift core rotation and is reported in Section 5.

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 40).

Figure 2 Ground floor Level 1 slab on grade (clockwise from top) a) View from Fire Service 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. Composite Metal Deck and Concrete Suspended Floor Slabs

The drawings specified a 200 mm thick Hi-Bond floor slab spanning north to south and seated on 400 mm wide precast log beams at 7500 mm centres (DENG Dwg S15 (Figure 57)). The slab was propped during construction and the trays pre-set upwards at approximately quarter points to a maximum of 20 mm at midspan and then the topping was instructed by the Specification (Clause 2.16 Figure 70) to be cast to provide the specified thickness.

Reinforcing mesh size 664 was specified under H12 saddle bars 4000mm long at the beams and draped to 20 mm above the Hi-bond at midspan.

The Base Metal Thickness (BMT) of the Hi-Bond is 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 p. 111).

The Hi-Bond decking was in all instances found to have fully 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 59)) as seen at Level 4 in Figure 22. The clamping action was sufficient for the decking to have fractured under tension during the collpase.

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 59) as seen in Items E14 (Figure 5) and E18 (Figure 11). On the edge beam Item E23 (Figure 6) 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 59) and Section 8 Dwg S15 (Figure 58)), as seen in Figure 9 and Figure 10.

c. Item E21: Architectural Cladding Panel

Figure 3 Pre-cast spandrel panel Item E21 (DENG Dwg S25 (Figure 65))

d. Shell Beam and Slab

i. Item E6

Figure 4 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 71).

ii. Grid 4/B-C : Item E14

Figure 5 Pre-cast shell beam (Item E14) from northern face Grid 4, west side of lift core (DENG B23 Dwg S18 (Figure 59)). (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 22 at Level 4 and Figure 23 at Level 3.

iii. Item E23

Figure 6 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 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. 400 mm Diameter Concrete Columns

i. Item E19

Figure 7 400 mm diameter column Item E19. (Left to right) a) Level 6 to Roof, likely location Grid F (Figure 57) 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 61).

ii. Item E33

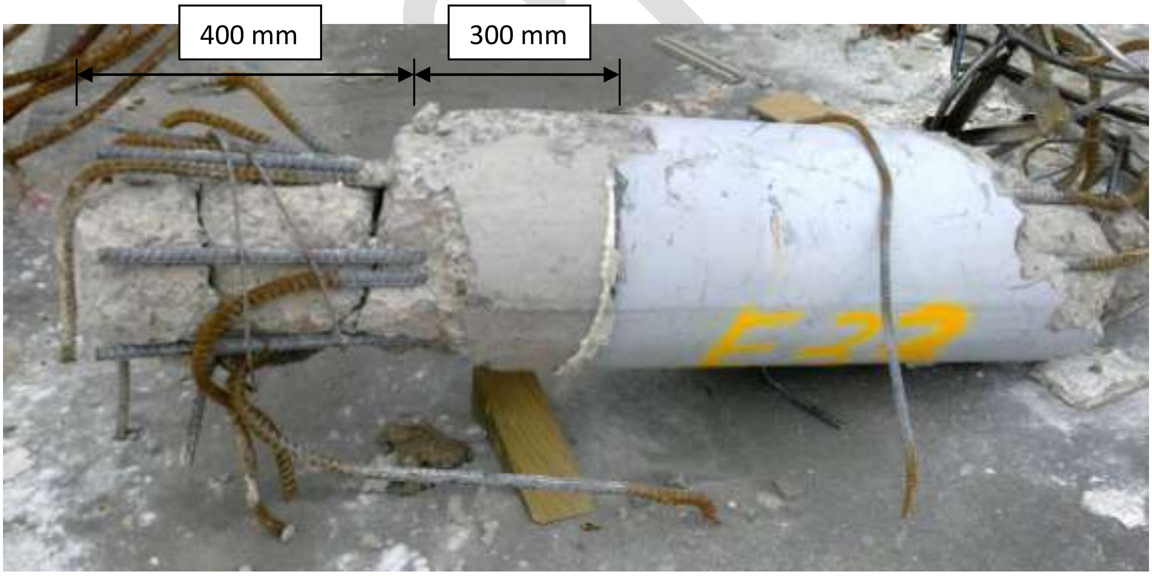


Figure 8 400 Diameter Exterior Column Item E33. (DENG C5 or C11, Dwg S15 (Figure 61)). Left end is bottom of column at floor level with concrete spalling over lapped vertical reinforcing. Horizontal cracking in core confined by R6 spiral which had fractured. The unpainted portion measured at 700 mm long was protected by spandrel panels (Figure 3, Figure 65 and Figure 66). Right-hand end fracture occurred below beam-column joint.

f. Line 2 and 3 Internal Precast Log Beams

i. Item E26

Figure 9 Interior Pre-cast Log Beam from Line 2 and 3 (DENG Section 3 Dwg S15 (Figure 58)) (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 10 Interior Pre-cast Log Beams from Line 2 and 3 (DENG Section 3 Dwg S15 (Figure 58)) 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 11 Item E18 Pre-cast edge beam north-west corner (DENG B22 Dwg S18 (Figure 59)) (from left to right) a) Smooth form finish at attachment to column 4A (DENG Detail 1 Dwg S19 (Figure 63)); b) No starters from pre-cast beam into slab to prevent the Hi-bond slab pulling away (DENG Section 4 Dwg S15 (Figure 58))

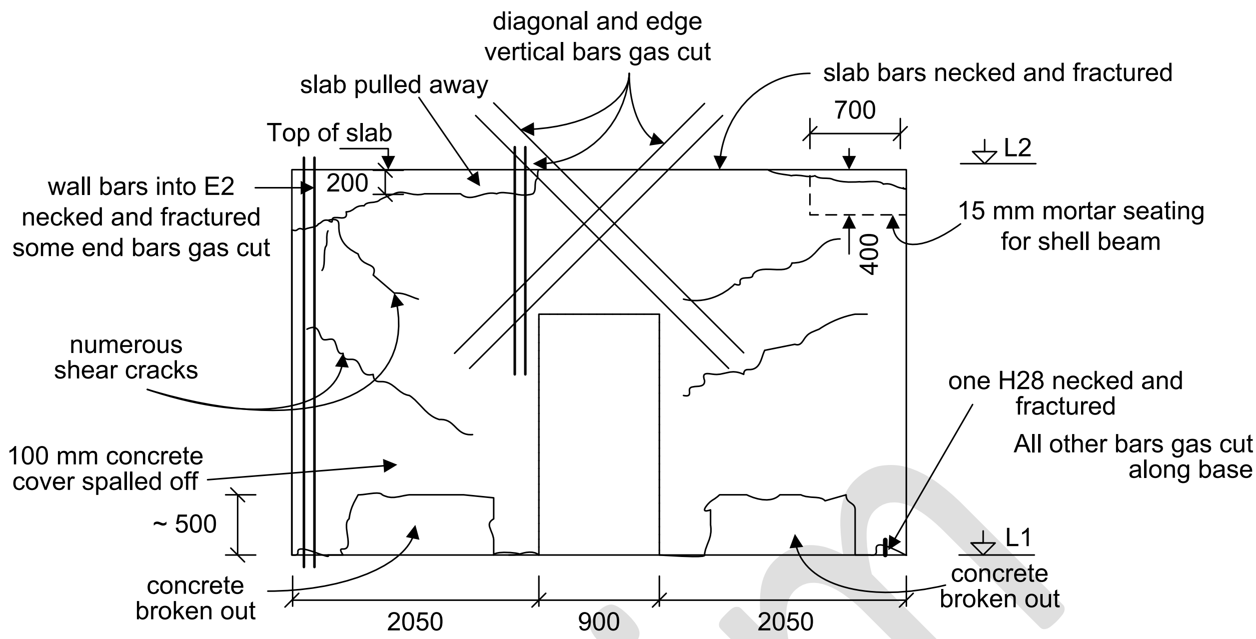
h. Line 1 Shear Wall

The Line 1 shear 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 67).

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

i. Line 1 Wall Level 1 to 2: Item E1

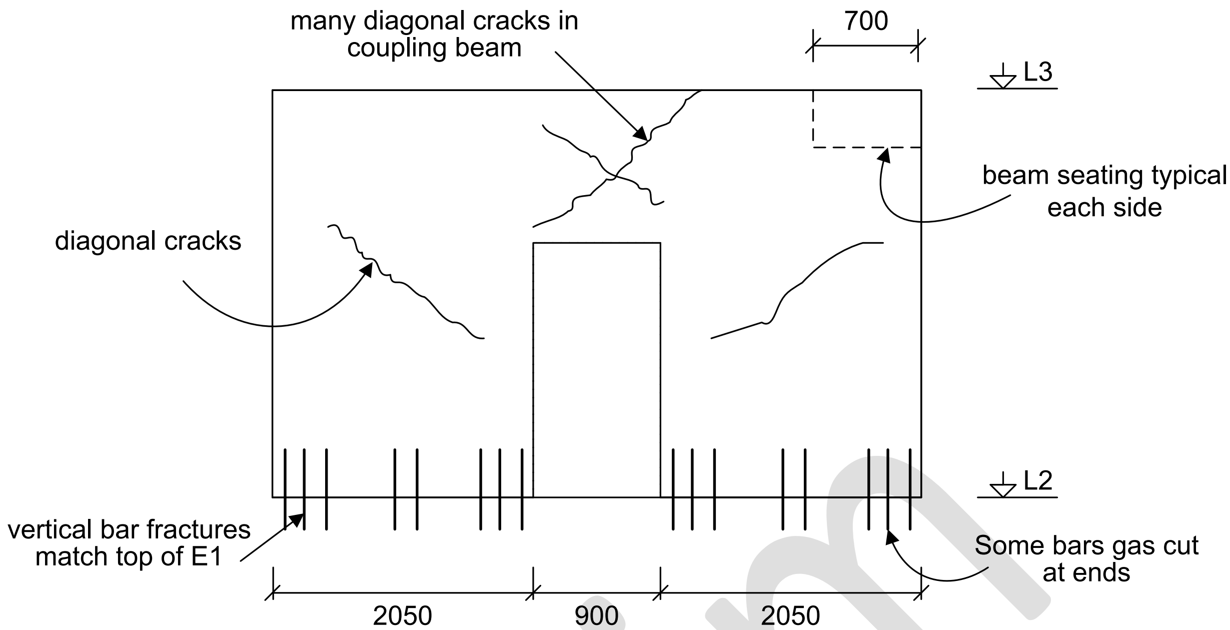
Figure 12 Line Shear Wall (Item E1) (clockwise from top left) a) Outer face of wall with lower portion of concrete removed during deconstruction exposing the reinforcing steel; b) Outer face with cracks highlighted by red paint; c) Inside face with cracks highlighted by red paint; c) Top west corner; d) Top east corner; e) Inside face of east pier


diagonal and edge
vertical bars gas cut
slab pulled away
slab bars necked and fractured
700
L2
Top of slab
200
wall bars into E2
necked and fractured
some end bars gas cut
15 mm mortar seating
for shell beam
400
numerous
shear cracks
one H28 necked and
fractured
All other bars gas cut
along base
100 mm concrete
cover spalled off
~ 500
L1
concrete
broken out
concrete
broken out
2050
900
2050

E1

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

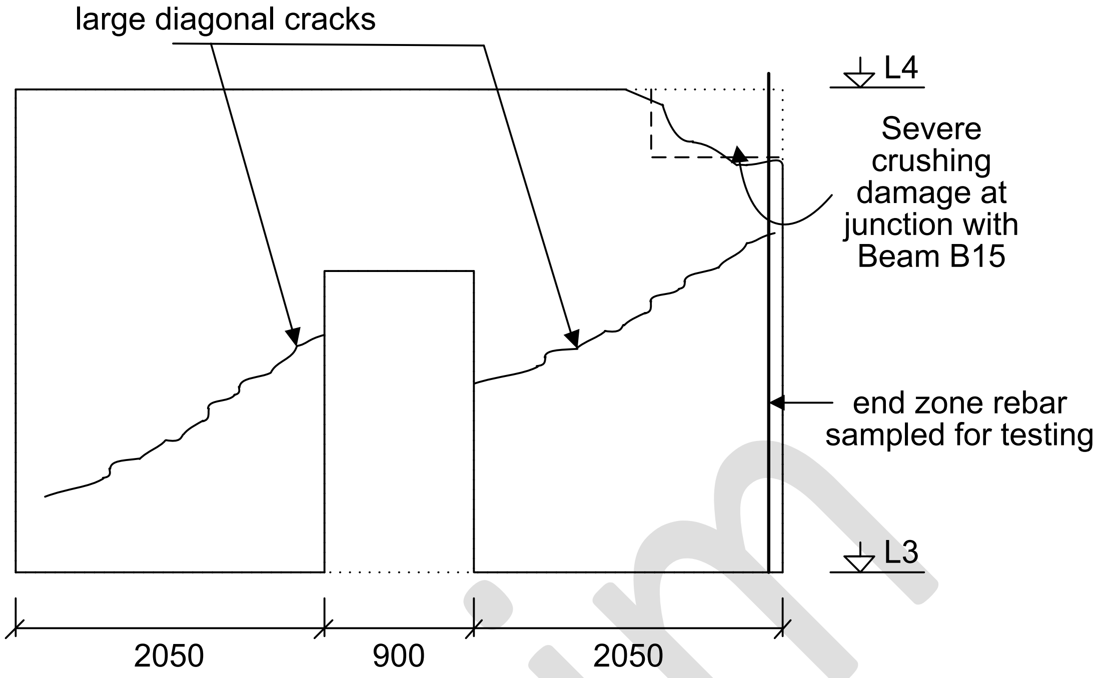
Figure 13 Line 1 Shear Wall Level 2 to Level 3 (Item E2) (clockwise from top left) a) Outside face with fire escape door attached. Cracks marked by red paint; b) Inside face of wall; c) east pier construction joint with necked and fractured bars indicated by red paint; d) Escape door edge of east pier 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.


many diagonal cracks in coupling beam
700
L3
beam seating typical each side
diagonal cracks
L2
vertical bar fractures match top of E1
Some bars gas cut at ends
2050
900
2050

E2

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

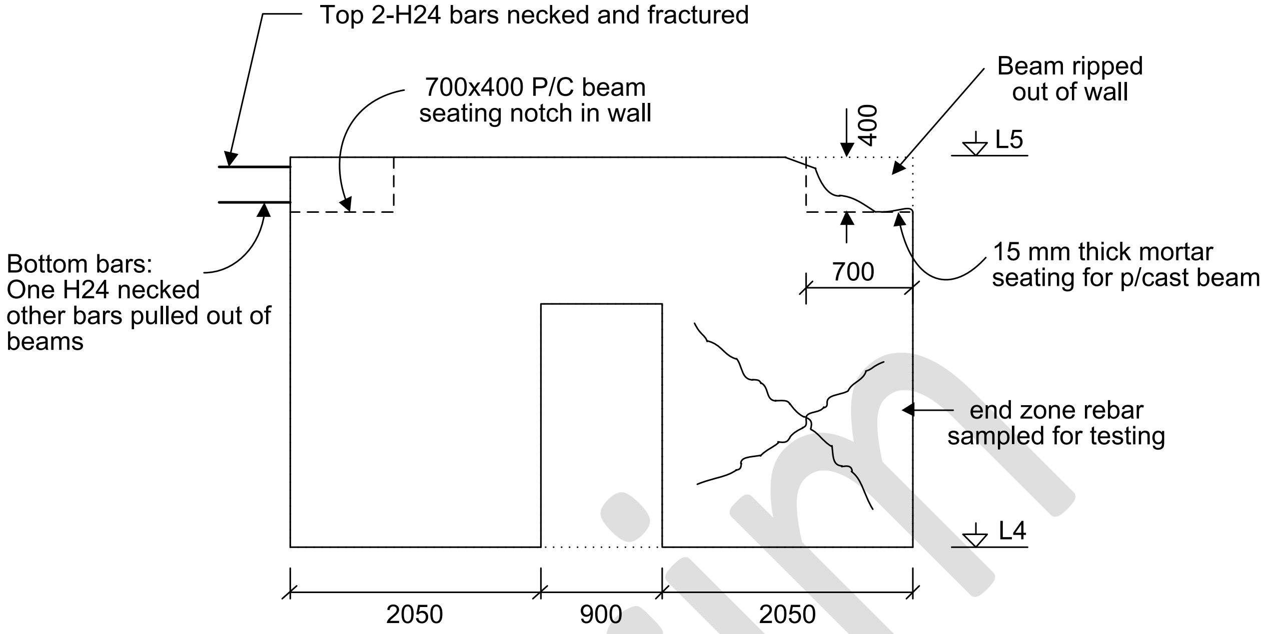
Figure 14 Line 1 Shear Wall Level 3 to Level 4 (Item E3) (clockwise from top left) a) Outer face; b) Inner face; c) Damaged top east corner; d) One way diagonal cracks running from bottom west to top east side marked by paint on outer face.


large diagonal cracks
L4
Severe crushing damage at junction with Beam B15
end zone rebar sampled for testing
L3
2050
900
2050

E3

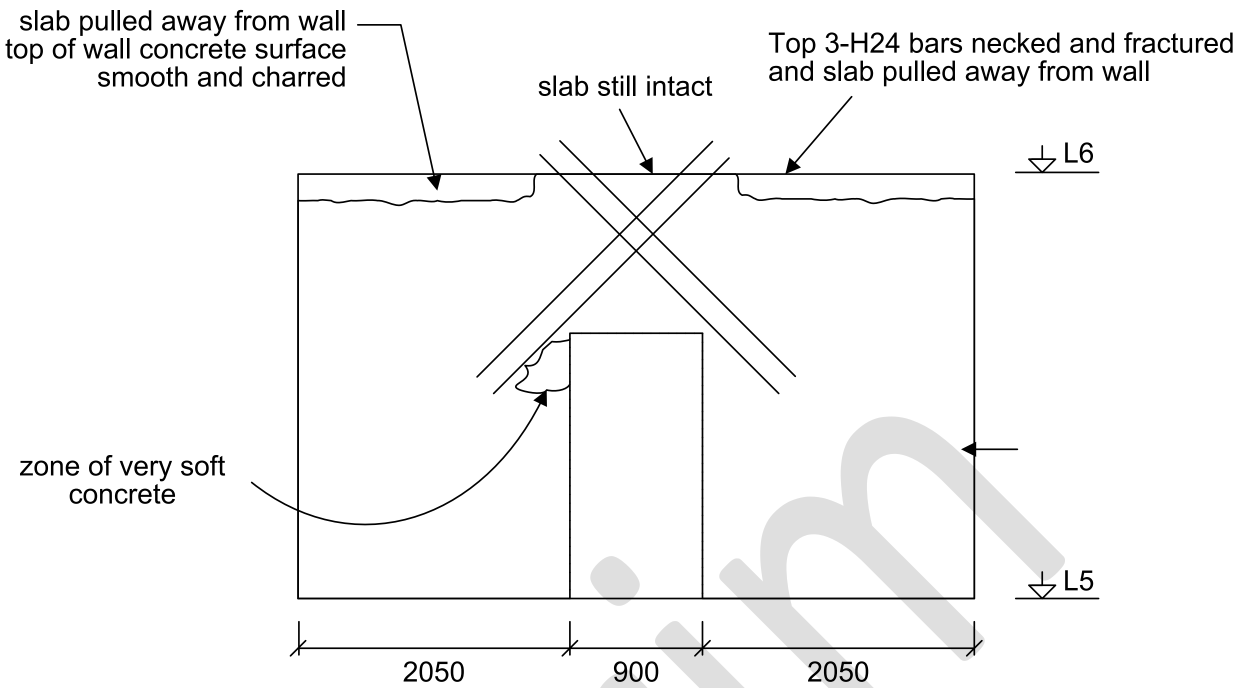
iv. Line 1 Wall Level 4 to 5: Item E4

Figure 15 Line 1 Shear 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 59)) and fire charring to spalled eastern edge; f) View from east to west of top east corner construction joint notch.


Top 2-H24 bars necked and fractured
700x400 P/C beam seating notch in wall
Beam ripped out of wall
400
L5
Bottom bars:
One H24 necked
other bars pulled out of beams
15 mm thick mortar seating for p/cast beam
700
end zone rebar sampled for testing
L4
2050
900
2050
E4

v. Line 1 Wall Level 5 to 6: Item E5

Figure 16 Line 1 Shear 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.


slab pulled away from wall
top of wall concrete surface
smooth and charred
slab still intact
Top 3-H24 bars necked and fractured
and slab pulled away from wall
L6
zone of very soft
concrete
L5
2050
900
2050

E5

i. Line 1 Wall Level 6 to Roof: Item E5A

Figure 17 Line 1 Shear 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. Line 5 Shear Wall

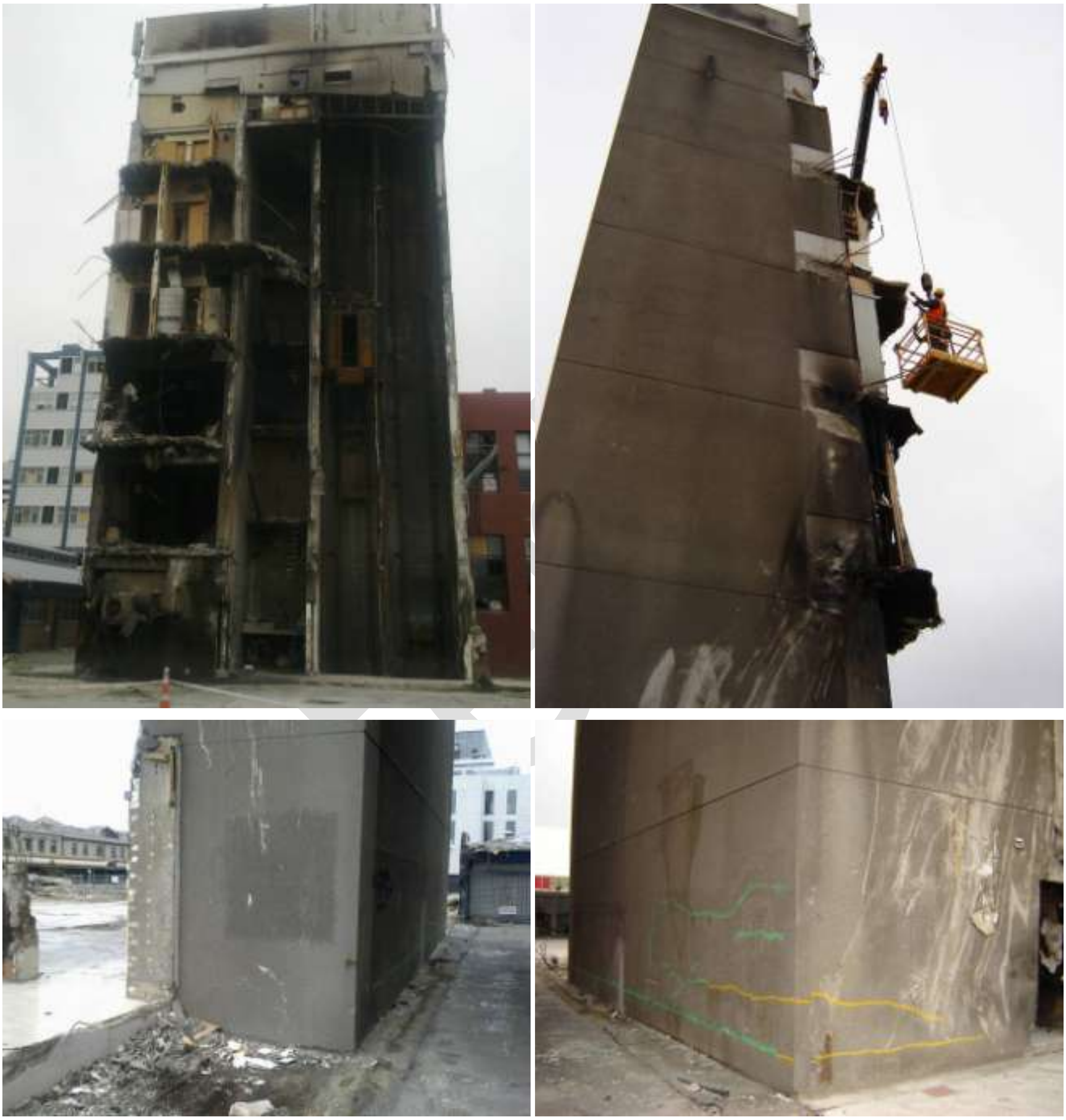


Figure 18 Line 4 to 5 Shear Core (DENG Dwg S15 (Figure 57)) (clockwise from top left) a) South face after site cleared with lift shaft for two cars on right, stair well in middle and amenity rooms on the left; b) West face; c) West and north face at Grid C/5 corner Level 1 to 2 with horizontal flexural cracks and construction joint identified by paint; d) East face with column C18 remnant at far left

Figure 19 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. Line 4 Stair and Lift Core Walls

The stair and lift core remaining walls, slabs and attachments were examined from a man-cage on 12th March, 2011 and from a N.Z. Fire Service snorkel platform on 5th April, 2011. Observations and comments are included in the captions to the photos.

i. Level 6 Slab Remnants

Figure 20 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 21 Line 4 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 eth drawings to be located near the top surface (DENG Dwg S16 (Figure 68)); (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 stair well. 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 59), Detail 5 S19 (Figure 64)), corresponding to the bent back bottom steel in shell beam Item E23 (Figure 6).

iii. Level 4 Slab Remnants

Figure 22 Line 4 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 59), Detail 5 S19 (Figure 64)), corresponding to the bent back bottom steel in shell beam Item E23 (Figure 6). b) Fractured vertical face of slab at stairwell wall, with fractured slab support beam (DENG B25 Dwg S18 (Figure 59)) top bar and charred fracture surface. c) Torn Hi-bond sheeting de-bonded from slab but still fixed in at pre-cast beam support.

iv. Level 3 Slab Remnants

Figure 23 Line 4 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 59), Detail 5 S19 (Figure 64)), corresponding to the bent back bottom steel in shell beam Item E23 (Figure 6). b) Ash on slab. Cored holes at fractured edge. c) Torn Hi-bond sheeting de-bonded from slab but still fixed in at pre-cast beam support (DENG B24 Dwg S18 (Figure 59)). d) Hi-bond deck and slab from below supported on beam B24

v. Level 2 Slab Remnants

Figure 24 Line 4 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 59), Detail 5 S19 (Figure 64)). Some diagonal beam–wall joint zone shear cracking can be seen in the wall end.

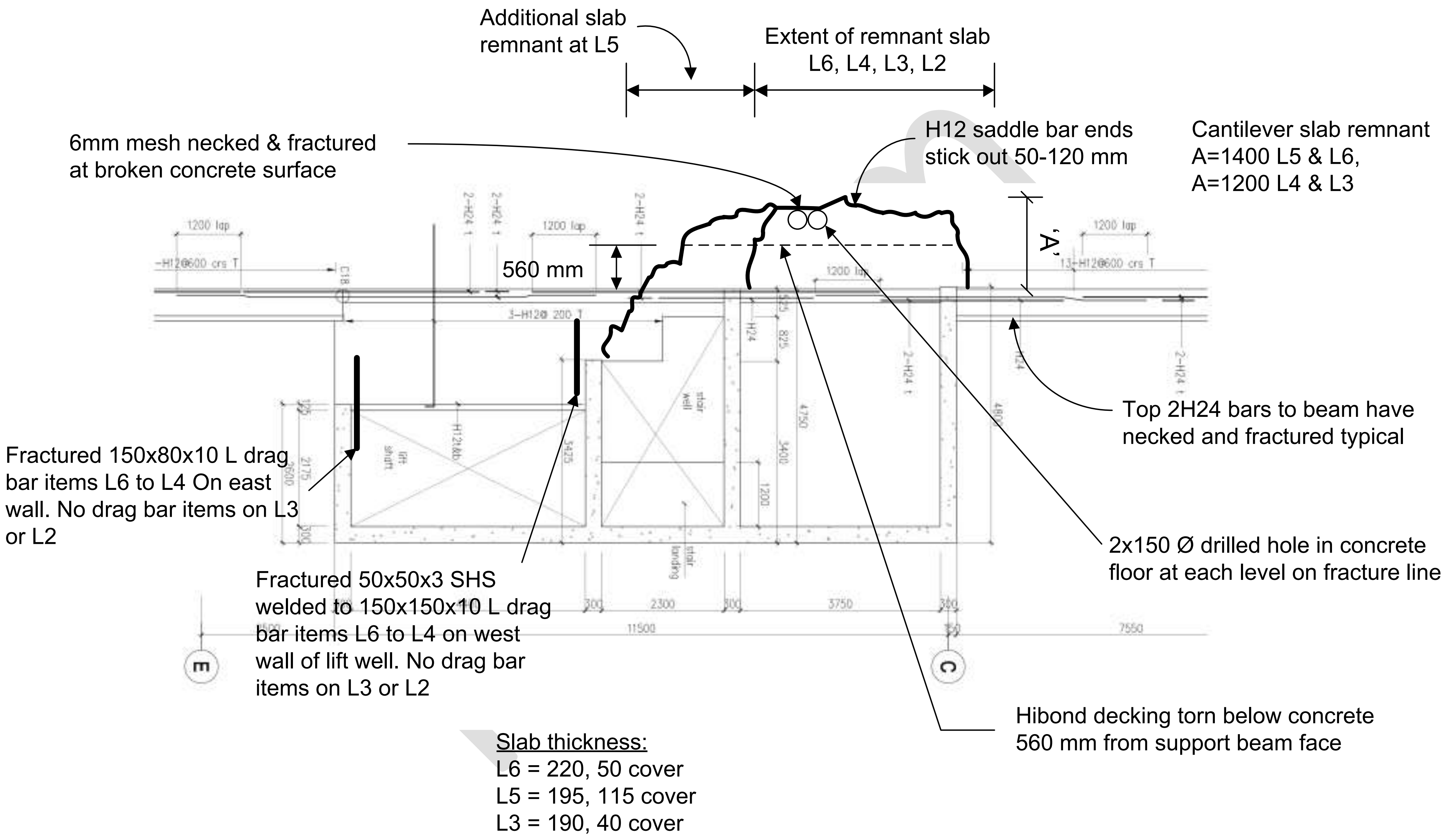


Figure 25 Extent of remaining slab at time of site examination. Portions of the slab had been removed during deconstruction for safety reasons.

l. Lift Core Wing Walls Diaphragm Connections

The Hi-bond floor slab at Level 4, 5 and 6 had additional drag bars connecting it to the north-south wing walls, on either side of the lift well, sometime after the original construction was completed. Bolts had been bolted through the slab and epoxy grout inserted to fill the gap between the bolts and the hole drilled into the slab as can be seen in Figure 29 to Figure 32.

Figure 26 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 25)

i. Level 2 Lift Well Wing Walls D and D.5

Figure 27 Level 2 Lift Well Wing Walls Grid D and D.5 (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.5

Figure 28 Level 3 Lift Well Wing Walls Grid D and D.5 (anti-clockwise from top left) a) Hi-bond 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 broken away as 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.5

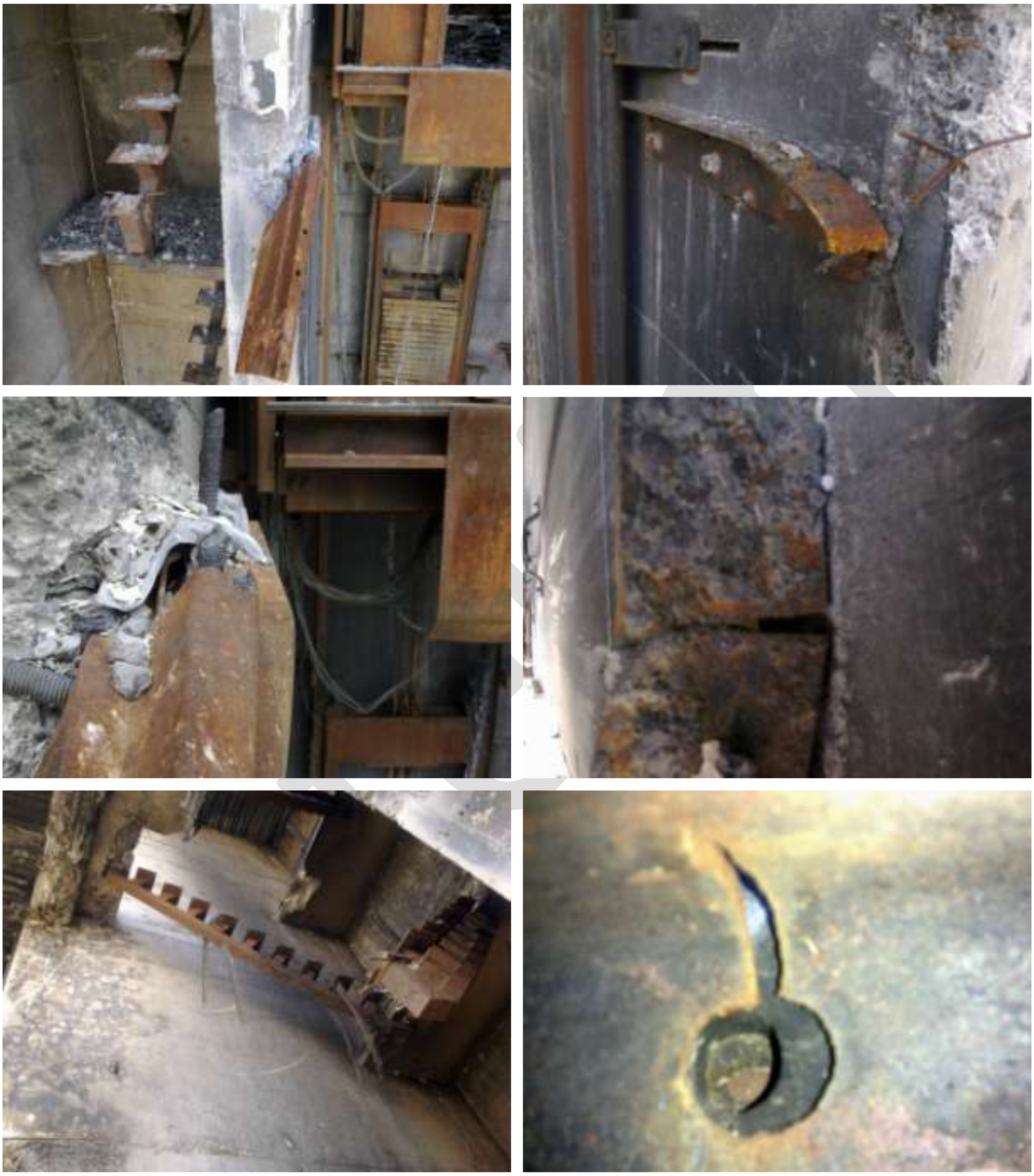


Figure 29 Level 4 Lift Well Wing Walls Grid D and D.5 (anti-clockwise from top left) a) 150x150x10 L with 50 x 3 SHS; 3 M24 bolts into wall and 6 –M20 bolts 350 mm long bolted through the slab at the Hi-bond rib with epoxy grout around bolt; b) Three M20 bolts remaining in the Grid D drag bar have fractured in tension at the underside of the bolt at the slab surface. The 50 x3 SHS has 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 69). d) Initiation of angle fracture at elongated hole without bolt into east wall. d) Fracture in angle running from corner of angle out towards toe viewed from above. f) Remnant of 150x80x10 L drag bar with 4 M24 bolts into grid D.5 wall. Angle has fractured in bending and tension two bolts in. End has been gas cut during de-construction.

Figure 30 M20 Drag Bar Bolt (left to right) a) Rusted portion had been through underside of steel drag bar. Grey portion with epoxy residue had been in slab; Diagonal fracture surface immediately below nut. (Portions shown are from two different bolts). b) Smooth diagonal surface to left indicative of tensile fracture in combination with some shear. Necked and dimpled fracture surface at underside of bolt (to the right) typical of direct tensile fracture in threaded rods.

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

Figure 31 Level 5 Lift Well Wing Walls Grid D and D.5 (anti-clockwise from top) a) 150x150x10 L with 50 x 3 SHS; 4-M24 bolts into wall and 6 –M20 bolts 350 mm long bolted through the slab at Hi-bond rib with epoxy grout around bolt; Three M20 bolts remaining in the Grid D drag bar have fractured in tension-shear at the underside of the bolt at the slab surface. The 50 x3 SHS has fractured in bending and tension at the bolt hole adjacent to last bolt into wall and twisted with the slab. b) Epoxy grout around bolt through slab; c) Holes for 3 M20 bolts through slab in twisted drag bar; d) 150x80x10 L drag bar with 5 M24 bolts into wall D.5. End of bar has been gas cut during de-construction.

v. Level 6 Lift Well Wing Walls D and D.5

Figure 32 Level 6 Lift Well Wing Walls Grid D and D.5 (anti-clockwise from top left) a) 150x150x10 L with 50 x 3 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 bolted through the slab at the Hi-bond rib with epoxy grout around bolt; These have fractured in tension-shear at the underside of the bolt at the slab surface and were measured with 110 mm stick-out above the item. The 50 x3 SHS has fractured in bending and tension at the bolt hole adjacent to last bolt into wall and the bar has twisted with the slab. b) Side view of drag bar remnant showing deck uplift at end bolt; c) Fracture surface of M20 bolt with smooth diagonal face indicative of tension-shear fracture; Epoxy grout around bolt through slab. f) End bolt with diagonal fracture and slab concrete remains; e) 150x80x10 L drag bar with 7- M24 bolts into east wall D.5. End of bar has been gas cut during de-construction.

m. Level 7 Lift Well Wing Wall D.5: Column C18 Connection

Figure 33 Lift Well Wing Wall D.5: Column C18 Connection (DENG Dwg S14 (Figure 61)); 3 x 20 to 24 mm diameter holes can be seen where reinforcing bars from column have pulled out. The drawing shows that 4-H20 bars were required to be bent in to the wall.

5. 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 gives heights and co-ordinates of the points shot with an accuracy of +/-5 mm.

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

Shots were initially taken on 14th April, 2010 to the approximate centres of the demolished remains of the concrete columns (Figure 51).

Shots were then taken on 18th 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 50).

Dumpy levels were subsequently taken 28th 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 allows 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 56)). 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 is consistent with measurements of flatness found on typical concrete floor slabs on grade in the United States which found a typical variation of +/- 16 mm in 3 metres. The NZS 3114 U3 surface finish criteria is much more severe at 3 mm over 3 metres but is known to be difficult to achieve and measure in normal construction such as this (Cowie and Hyland 2008).

NZS 3109 sets a tolerance of +/- 5 mm on the thickness of the floor slab. The overall level of a slab cast to level is able to vary by +/- 12 mm where the nearest surface above it is between 3 and 6 m from it (SNZ 2003).

The DENG specification section 2.8 (Figure 70) required the floor slab to achieve a levelness tolerance of +/- 15 mm and flatness of +/-6 mm over 3 m

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 is 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 is 89 mm.

d. Core Wall Lean

The core walls on Line 5 were found to have a northwards lean of 91 mm over 18.53 m between Level 1 and Level 7 at the eastern end, and 68 mm over 18.53 m at the western end (Figure 50).

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

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 earthquake generally consistent with normal international construction practice and close to reasonable and specified tolerances for this form of construction for car park slabs on grade.

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

As a consequence it is concluded that no slab or foundation settlement can be inferred to have occurred as a result of the earthquake.

The northwards lean in the Line 5 shear wall is concluded to have occurred during construction as:

1. No evidence of settlement 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 6.

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 6.

6. Line 4 and 5 Lift Core Foundation Inspection

The slab was removed and a pit dug adjacent to the northwest corner of the core walls (Grid C/5) on 10th May 2011, to look for damage in the foundation beams around the lift core area and signs of liquefaction (Figure 52).

Another pit was subsequently dug adjacent to the northwest corner of the lift core after the walls had been substantially demolished on 13th May 2011. This was to check the side of the foundation for cracking after remains of rotted timber boxing had been removed and there was no danger from falling debris from the lift core to those undertaking the inspection.

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

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

7. Reinforcing Steel Properties

a. Sample Locations

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

i. H16: Line 1 Wall Level 1 Door Infill

Figure 34 Line 1 Wall Item E1 H16 bars from masonry door infill.

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

Figure 35 H28 from east and west end of Line 1 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 Wall Ends Level 3 (E3) and 4 (E4)

Figure 36 Locations of H24 reinforcing bar samples (left to right) a) East end of Line 1 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, 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 has elongation at maximum force Agt 3.3% less than that of the H28 bar extracted higher up the wall on the west side E1W. It also has a measured yield stress of 464 MPa which is 17 MPa higher. This indicates that the E1E bar has 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 has 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 for chemical analysis. The analyses were conducted on 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 Group metallurgist advised that the results are consistent with them being from the same or similar production runs and are within the variances expected from product testing.

The chemical analyses show the bars 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 Group laboratory

8. Concrete Properties

a. Drilled Concrete Core Properties

Concrete cores were extracted from suspended slabs in two locations (Item E14 and E23); the Line 5 shear wall at Level 1; the Line 1 shear wall between Level 4 and 5 (Item E4) ; the 400 mm diameter column between Level 6 and 7 (Item E25) (Figure 39); and the Level 1 column C18 stub (Figure 40).

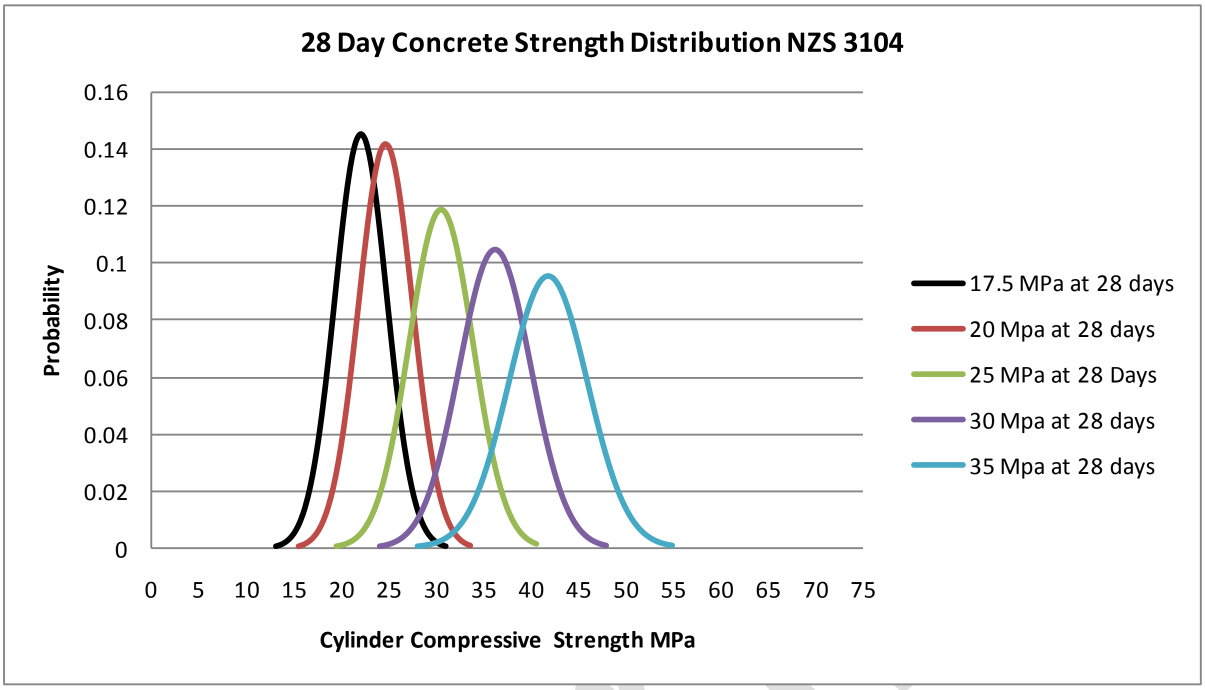
Concrete compressive testing was undertaken for slabs, beams and columns by Opus International Consultants Christchurch Laboratory (Jones 2011). Concrete compressive and chord modulus of elasticity was undertaken for shear wall cores at Central Laboratories in Wellington (Wong 2011).

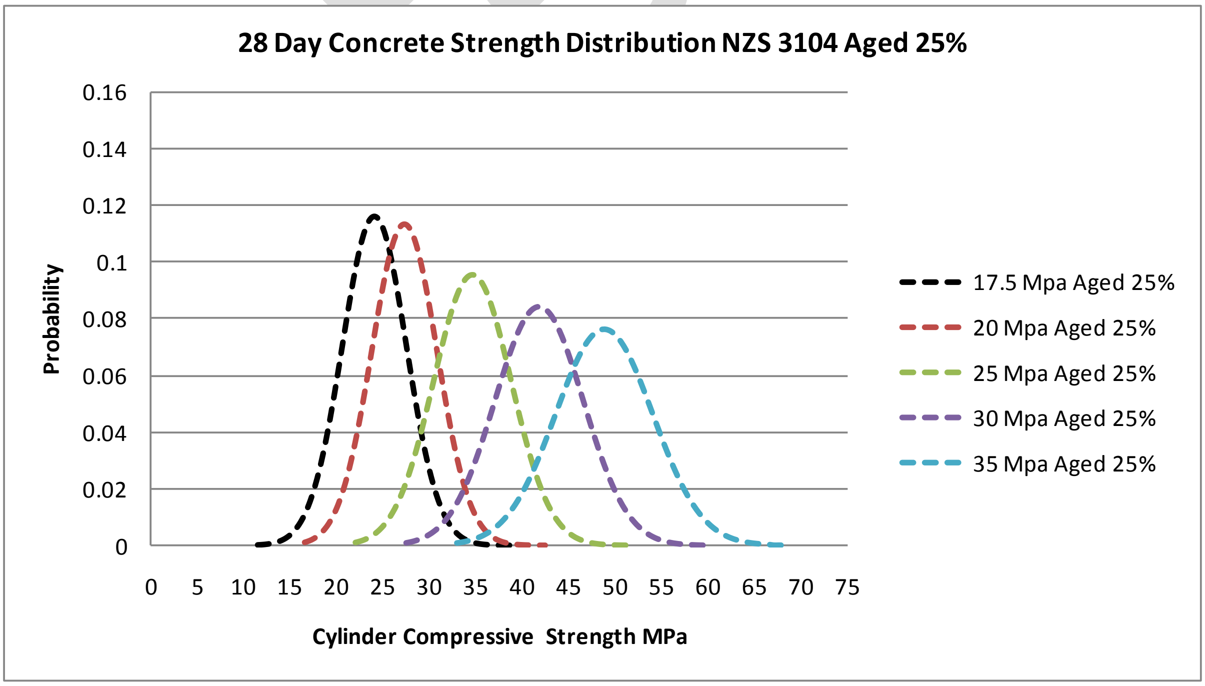
b. Allowance for Strength-aging Effect of Concrete

Concrete is known 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 Caltrans found in California that concrete with 20 to 25 MPa specified 28 day strength had at least 25% strength – aging over 20 to 30 years. Concrete batching practice typically sought to achieve a target strength 20% greater than the specified 28 day cylinder compressive strength. This led to the use of a divisor of 1.5 on the strength-aged specimen test results to approximate the specified 28 day compressive strength or 1.25 for strength–aging alone (Priestley, Seible et al. 1996).

The long term statistical relationships of New Zealand concrete properties at 28 days for specified concrete grades are published in the concrete production standard NZS 3104 Table 2.5A (SNZ 2003),(SNZ 1983). The statistical properties of the same concrete strength-aged have been derived by application of a factor of 1.25 to the mean and standard deviation of the 28 day strength properties (Table 3).

The lower bound 5% and 0.1% confidence limits on the sample means of aged concrete test properties have been derived for various sample sizes to allow statistical assessment of conformance with the originally specified 28 day concrete strength.


28 Day Concrete Strength Distribution NZS 3104
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
Probability
0 5 10 15 20 25 30 35 40 45 50 55 60 65 70 75
Cylinder Compressive Strength MPa
17.5 MPa at 28 days
20 Mpa at 28 days
25 MPa at 28 Days
30 Mpa at 28 days
35 Mpa at 28 days


28 Day Concrete Strength Distribution NZS 3104 Aged 25%
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
Probability
0 5 10 15 20 25 30 35 40 45 50 55 60 65 70 75
Cylinder Compressive Strength MPa
17.5 Mpa Aged 25%
20 Mpa Aged 25%
25 Mpa Aged 25%
30 Mpa Aged 25%
35 Mpa Aged 25%

Concrete Specified Grade Properties at 28 days and Strength-aged by 25%						
Variability of 28 day cylinder strength from Table 2.5A NZS3104						
Specified 28 day Strength	**Lower 5%**	**17.5**	**20.0**	**25.0**	**30.0**	**35.0**
	Lower 0.1%	13.6	16.1	20.3	24.7	29.2
	Mean	22.0	24.6	30.5	36.2	41.9
	cov	0.125	0.114	0.110	0.105	0.100
	standard deviation	2.75	2.82	3.35	3.80	4.18
Strength-aged by 25%	**Lower 5%**	**21.9**	**25.0**	**31.2**	**37.5**	**43.7**
	Lower 0.1%	17.1	20.1	25.4	30.9	36.4
	Upper 95%	33.2	36.6	45.0	53.1	60.9
	Upper 99.9%	38.0	41.5	50.9	59.7	68.2
	Mean	27.5	30.8	38.1	45.3	52.3
	cov	0.125	0.114	0.110	0.105	0.100
	standard deviation	3.44	3.52	4.19	4.75	5.22
Sample Mean Limits n=36	**Lower 5%**	**26.6**	**29.8**	**37.0**	**44.0**	**50.9**
	Lower 0.1%	25.8	29.0	36.0	42.9	49.7
Sample Mean Limits n=19	**Lower 5%**	**26.2**	**29.5**	**36.5**	**43.5**	**50.3**
	Lower 0.1%	25.1	28.3	35.2	42.0	48.7
Sample Mean Limits n=13	**Lower 5%**	**25.9**	**29.2**	**36.2**	**43.1**	**49.9**
	Lower 0.1%	24.6	27.8	34.6	41.3	47.9
Sample Mean Limits n=7	**Lower 5%**	**25.4**	**28.6**	**35.5**	**42.3**	**49.1**
	Lower 0.1%	23.6	26.7	33.3	39.8	46.3
Sample Mean Limits n=6	**Lower 5%**	**25.2**	**28.4**	**35.3**	**42.1**	**48.8**
	Lower 0.1%	23.2	26.4	32.9	39.4	45.8
Sample Mean Limits n=4	**Lower 5%**	**24.7**	**27.9**	**34.7**	**41.4**	**48.0**
	Lower 0.1%	22.3	25.4	31.8	38.1	44.4
Sample Mean Limits n=3	**Lower 5%**	**24.3**	**27.4**	**34.2**	**40.8**	**47.4**
	Lower 0.1%	21.5	24.6	30.8	37.0	43.1
Sample Mean Limits n=2	**Lower 5%**	**23.5**	**26.7**	**33.3**	**39.8**	**46.3**
	Lower 0.1%	20.1	23.2	29.1	35.1	41.1

Table 3 Specified grade properties of concrete at 28 days in accordance with NZS3104 and strength-aged by 25%

c. Suspended Slab Compressive Test Properties

Average compressive strength from the six cores for the slabs attached to Items E14 and E23 was 24.7 MPa, with a minimum of 19.5 and maximum of 30.5 MPa.

The specified concrete for the slabs was 'high grade' in accordance with NZS3109:1980 (SNZ 1987), with a compressive strength was fc' = 25 MPa at 28 days (Figure 70).

The sample mean of 24.7 MPa of the suspended slab concrete is less than the lower 5% confidence limit of 25.2 MPa but greater than the lower 0.1% confidence limit of 23.2 MPa for concrete with 28 day strength of 17.5 MPa strength-aged by 25% (Table 3).

In conclusion, on the basis of 25% strength-aging at the time of the tests, the suspended slab concrete would not have complied with the requirements of concrete with the specified 28 day strength of 25 MPa.

It is also did not comply with the requirements for concrete with 28 day strength of 17.5 MPa with an acceptable level of confidence (Figure 37).

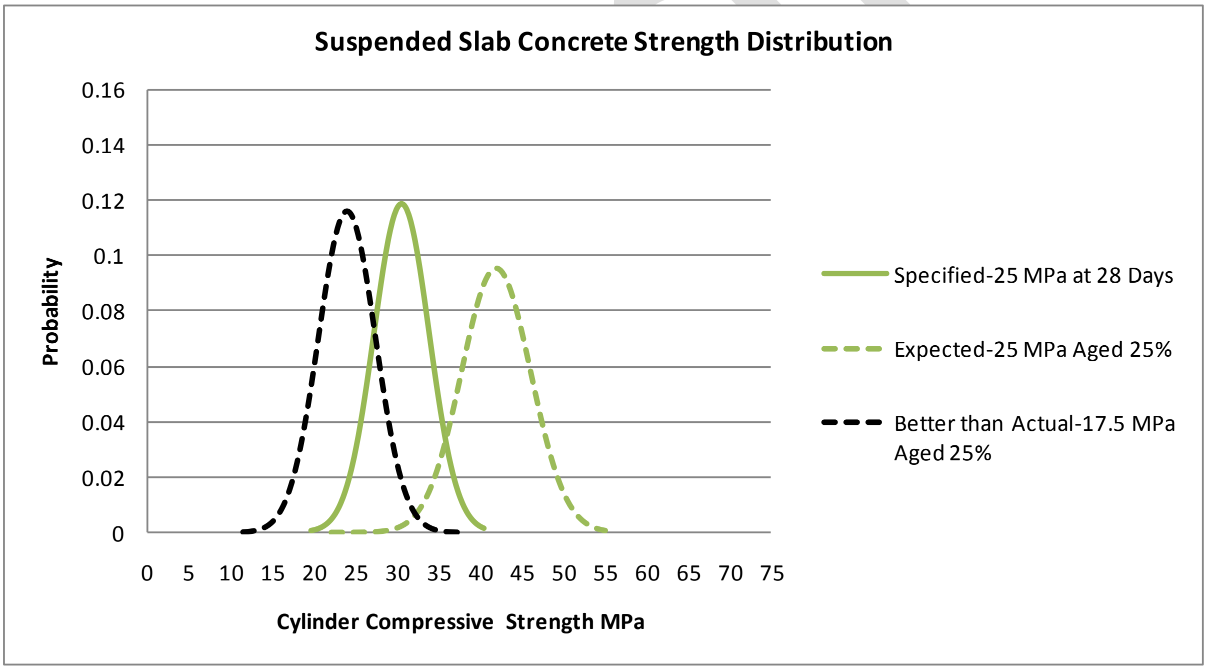


Figure 37 Suspended slab concrete properties

d. Shear Wall Line 1 and 5 Concrete Test Properties

i. Shear Wall Compressive Test Properties

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, with a minimum of 30.0 and maximum of 39.5 MPa.

The specified concrete for the walls was 'high grade' in accordance with NZS3109:1980, with a compressive strength was fc' = 25 MPa at 28 days (Figure 70).

The sample mean of the shear wall concrete is less than the lower 5% confidence limit of 35.5 MPa for concrete with the specified 28 day strength of 25 MPa strength-aged by 25% (Table 3).

The sample mean of the shear wall concrete is greater than the lower 5% confidence limit of 28.6 MPa for concrete with 28 day strength of 20 MPa strength-aged by 25% (Table 3).

In conclusion on the basis of 25% strength–aging at the time of the tests, the shear wall concrete would not have complied with the requirements of concrete with the specified 28 day strength of 25 MPa with an acceptable level of confidence.

However it would have complied with the requirements for concrete with 28 day concrete strength of 20 MPa with an acceptable level of confidence (Figure 38).

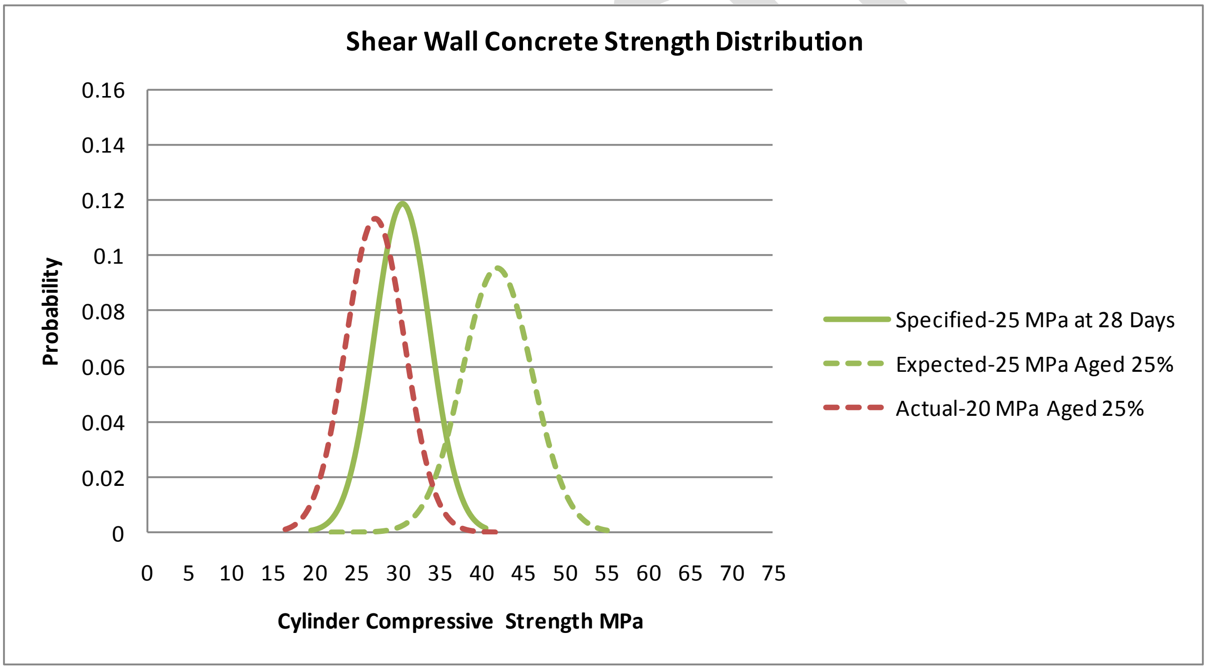


Figure 38 Shear Wall Concrete Properties

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.

e. Compressive Test Properties of Concrete Column Remnants at CTV Site

i. Level 6 400 mm Diameter Column

Average compressive strength from the three cores for the Level 6 column (Item E25) was 23.3 MPa, with a minimum of 16.0 and maximum of 27.5 MPa. If the lower result of the three is ignored as an outlier the mean is 27.0 MPa

The specified concrete for the columns at and above Level 3 was 'high grade' in accordance with NZS3109:1980, with a compressive strength was fc' = 25 MPa at 28 days (Figure 70).

The sample mean of 23.3 MPa for 3 tests of the 400 mm diameter column is less than the lower 5% confidence limit of 27.4 MPa for concrete with specified 28 day strength of 20 MPa strength-aged by 25% (Table 3). However for the two higher results the sample mean is greater than the lower 5% confidence limit of 26.7 MPa for concrete with specified 28 day strength of 20 MPa strength-aged by 25%.

The minimum test value for the 400mm diameter column of 16.0 MPa is less than the single sample 0.1% lower bound test limit of 17.1 MPa for concrete with 28 day strength of 17.5 MPa strength-aged by 25%. But this may be considered to be an outlier result caused by cracking or similar effects during extraction.

In conclusion on the basis of 25% strength-aging at the time of the tests, the Level 6 to Roof 400 mm diameter column concrete would not have complied with the specified requirements for concrete with 28 day strength of 25 MPa.

However it may have complied with the requirements of concrete with specified 28 day strength of 20 MPa if the lowest test reading is treated as an outlier and ignored.

ii. Level 1 400 mm Square Column C18

Average compressive strength from the six cores tested for the Level 1 square column C18 (DENG Dwg S9 (Figure 56) and S14 (Figure 60 and Figure 61)) was 16.0 MPa, with a minimum of 11.0 and maximum of 25.1 MPa.

The specified concrete for the columns founded at Level 1 was 'high grade' in accordance with NZS3109:1980, with a compressive strength was fc' = 35 MPa at 28 days (Figure 70).

The sample mean of the Level 1 400mm square column concrete is less than the lower 5% confidence limit of 25.2 MPa for concrete with 28 day strength of 17.5 MPa strength-aged by 25% (Table 3).

The minimum test value for the 400 mm square column concrete of 11.0 MPa is less than the single sample 0.1% lower bound test limit of 17.1 MPa for concrete with 28 day concrete strength of 17.5 MPa strength-aged by 25%.

The test samples have been retained so that chemical testing can be undertaken if needed to confirm if the samples had been affected by heat from the fire that occurred after the collapse.

In conclusion, on the basis of 25% strength-aging 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 17.5 MPa with an acceptable level of confidence.

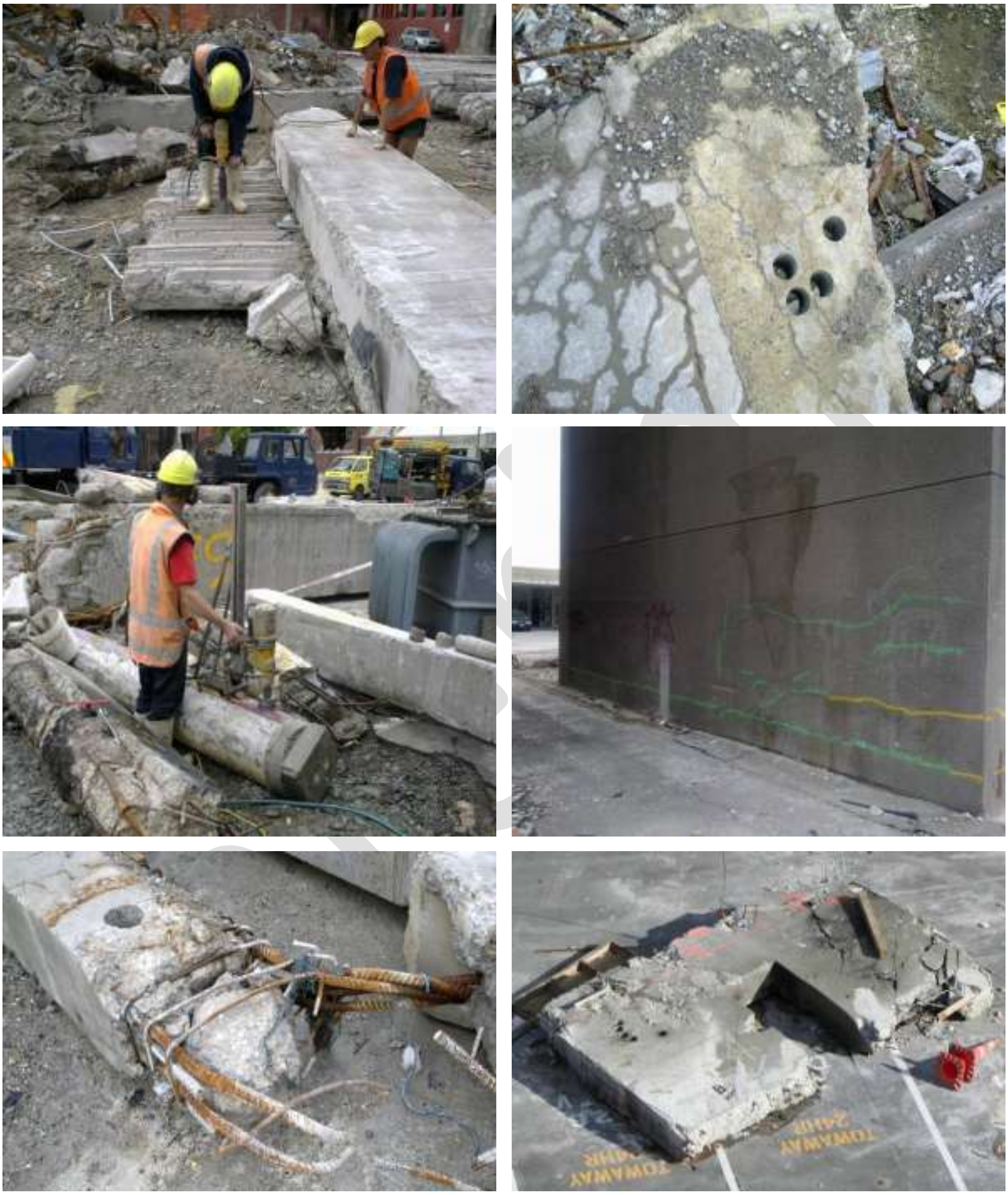


Figure 39 Concrete core locations (clockwise from top left) a) Slab cores from Item E23; b) Slab cores from Item E14; c) Line 5 wall cores at Level 1 centre stair well area; d) Line 1 wall cores from Item E4 Level 4 to Level 5 in west pier; e) Pre-cast log beam core in side; f) 400 mm Diameter Column E25 Level 6 to roof cores.

Figure 40 400 mm Square Column C18 at Grid 4/D.5 adjacent to lift core walls showing compressive failure (left to right) a) Spear head shape of failure surface indicative of compressive failure. Fire scorching to surface from smouldering fire; b) Holes showing locations of drilled cores extracted for compression testing.

f. Compressive Test Properties of Concrete Column Remnants at Landfill

i. Description of Column Remnants at Landfill

Thirteen 400 mm diameter 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 42).

The columns came 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.

Column Item E25 that had previously been cored at the CTV site was among those found.

The bottom of columns could be identified by the terminating vertical lap bars.

ii. Circular Column Remnants

Circular column remnants test Items C7, C8, C9, C11, C12 and C13 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 46 and Figure 8). In that case spear head style shear or flexural hinging commenced approximately 1350 mm above the base and terminated at around 1600 mm. This appears to coincide with the end of the column lap bars which were specified to be 1200mm long (DENG Dwg S14 Figure 61).

Column remnant C6 also had column mid-height failure though the top of it was connected into the roof. Column remnant C3 has 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 3 and Figure 65). The 400 mm diameter columns that suffered the mid-height failures may therefore have been from Grids 1, 4 and F like Item 33.

Figure 41 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.

Circular column remnants C2 and C10 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 C10 had a downpipe cast into it (Figure 47).

The specified concrete strength for these Level 1 columns was 35 MPa at 28 days according to the Specification. However the inferred strengths show that the concrete used was consistent with 17.5 MPa 28 day strength concrete aged by 25% (Table 4 and Table 6). 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 41).

Circular column remnants C1, C4 and C5 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 C1 was a full height column from Level 6 to Roof with hinging at its base and still connected by reinforcing steel to column remnant C8 which had hinging at its base and failure also at mid-height, with all the concrete in between gone.

iii. Rectangular Columns Line A

Columns R1, R2, R3, R4and R4', and R5 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 R4 and R4' were lower and upper columns running between Level 5 to Level 6 and the Roof respectively still connected by reinforcing steel.

Columns R7 and R6, and R8 and R9 were also lower and upper columns respectively running between two unknown levels still connected by reinforcing steel through failed beam-column joint zones.

Columns R6, R7, R8, R10 and R10' had beam-column joint failures at the base and mid-height hinging.

The bottom of the Column R9 had a smooth flat surface as would have been obtained from an unroughened construction joint at floor level.

Figure 42 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.

iv. Rebound Hammer Testing and Coring to ASTM C805

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

Two cores were subsequently extracted and tested from each of five column test locations that had 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 (Figure 54).

v. Inferred Strengths and Comparison to Aged 28 Day Grade Statistics

The compressive strengths at each location were inferred using the 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 (Figure 54).

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, as the strength to hammer number relationship varies with concrete mixes (cl. 5.2 ASTM C805). The charts however do provide a useful 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.

The statistical parameters of the samples tested were compared to those of 28 day strength concrete manufactured in accordance with NZS3104 and adjusted for 25% aging (Table 4).

Figure 43 400 mm diameter columns full height (left to right) a) Test Item C1 Level 6 to roof column with base hinging failure, still connected by reinforcing to C8 below; b) C4 hinging top and bottom; c) C5 Level 6 to roof column with hinging at base.

Sample		Tested		Best Fit	
		Sample Statistics		28 day strength with 25% strength Aging	
Specific Level 5 & 6 Columns		Mean	27.6	27.5	Mean Aged Strength
C1, C5,C6,C9,R2,R4,R4'		SD	5.0	3.44	
(Rebound Hammer tests)		n	13	25.9	Sample mean lower 5% limit
		Max	38.2	38.0	Upper 99.9% Aged Strength
		Min	20.8	17.1	Minimum 0.1% AgedStrength
Specified 28 Day Strength	**25**	Lower 5%	19.39	21.9	Lower 5% Aged Strength
		cov	0.18	**17.5**	**Inferred 28 Day Strength**
Other Level 3,4 & 5 Columns		Mean	25.3	27.5	Mean Aged Strength
C3, C7, C8,C11, C12, C13,		SD	5.2	3.44	
R1, R3, R5, R6, R8, R9, R10'		n	13	25.9	Sample mean lower 5% limit
(Rebound Hammer tests)		Max	33.2	38.0	Upper 99.9% Aged Strength
		Min	18.5	17.1	Minimum 0.1% AgedStrength
Specified 28 Day Strength	**25**	Lower 5%	16.77	21.9	Lower 5% Aged Strength
		cov	0.20	**17.5**	**Inferred 28 Day Strength**
Assumed Level 2 Columns		Mean	43.7	45.3	Mean Aged Strength
C4, R7, R10		SD	3.6	4.75	
(Cores tests C4 and R7)		n	4.0	41.4	Sample mean lower 5% limit
		Max	47.8	59.7	Upper 99.9% Aged Strength
		Min	39.5	30.9	Minimum 0.1% AgedStrength
Specified 28 Day Strength	**30**	Lower 5%	37.8	37.5	Lower 5% Aged Strength
		cov	0.08	**30.0**	**Inferred 28 Day Strength**

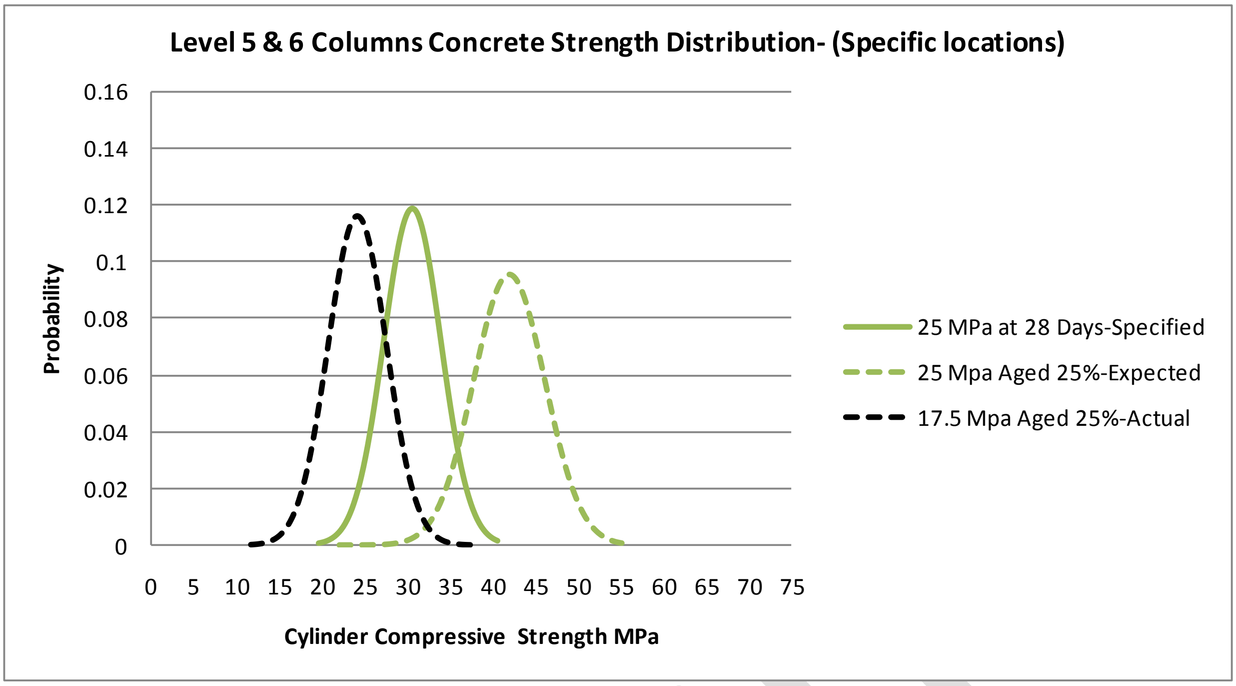
Table 4 Inferred strengths and statistical parameters of columns tested compared to 17.5 MPa and 30 MPa 28 day strength concrete aged by 25% and sample mean lower 95% acceptance limits.

vi. Column Properties at Landfill

Columns able to be specifically identified as being from Level 5 and 6 had a sample mean of 27.6 MPa which is greater than the lower 5% confidence limit of 25.9 MPa for concrete with specified 28 day strength of 17.5 MPa aged by 25% (Table 4).

There was a marked difference in the tested core strengths of columns C4 (46.6 MPa) and R7 (40.9 MPa) and the other columns core tested (23.9 MPa). R7 was the lower column of two, still attached by reinforcing steel to Column R6. Column R6 had significantly lower core test strengths (25.5 MPa) than R7. Column R10 had a rebound hammer number (HN=50) similar to Column C4 (HN=48.5) . It is therefore considered that columns C4, R7 and R10 were cast using concrete with different specified 28 day strength than the others in the sample tested. The average of the 4 cored test results from C4 and R7 is 43.7 MPa which is greater than the lower 5% confidence limit of 41.4 MPa for concrete with specified 28 day strength of 30 MPa aged by 25% (Figure 45).

Column remnants that could not be specifically identified as being from Level 5 or 6 had a sample mean of 29.1 MPa which is greater than the lower 5% confidence limit of 26.2 MPa for concrete with specified 28 day strength of 17.5 MPa aged by 25% (Figure 44). This excluded the Level 1 entry column remnants C2 and C10, and the higher strength column remnants C4, R7 and R10.



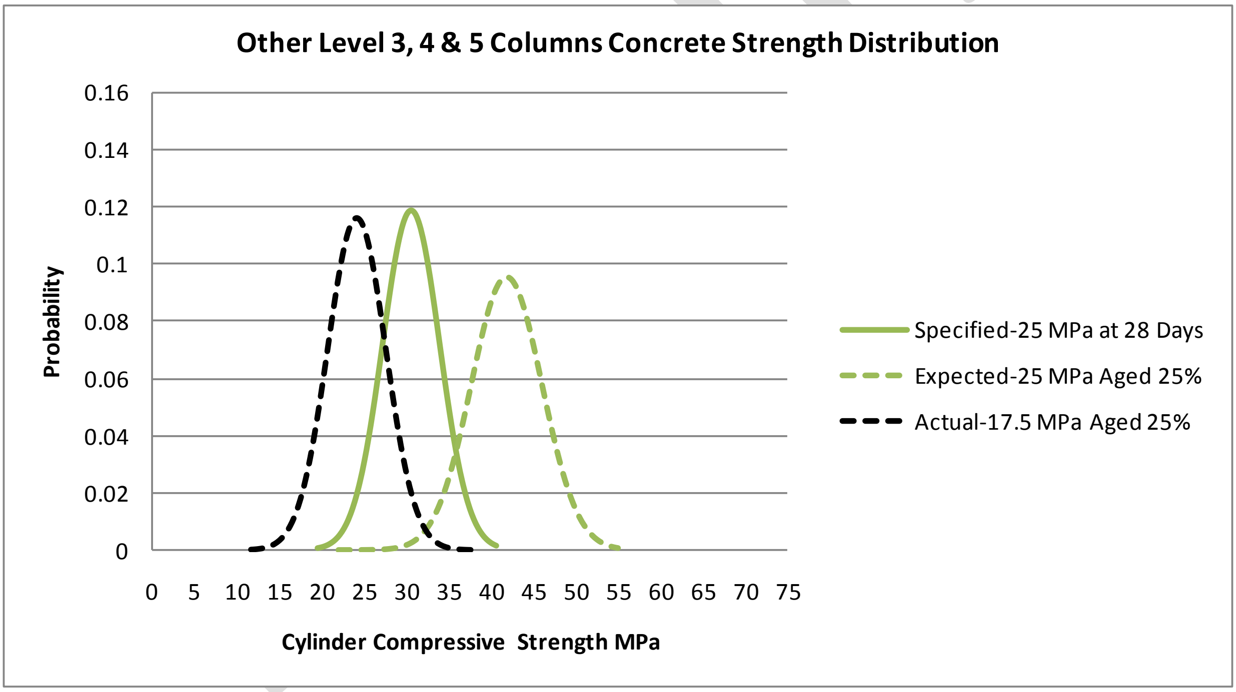


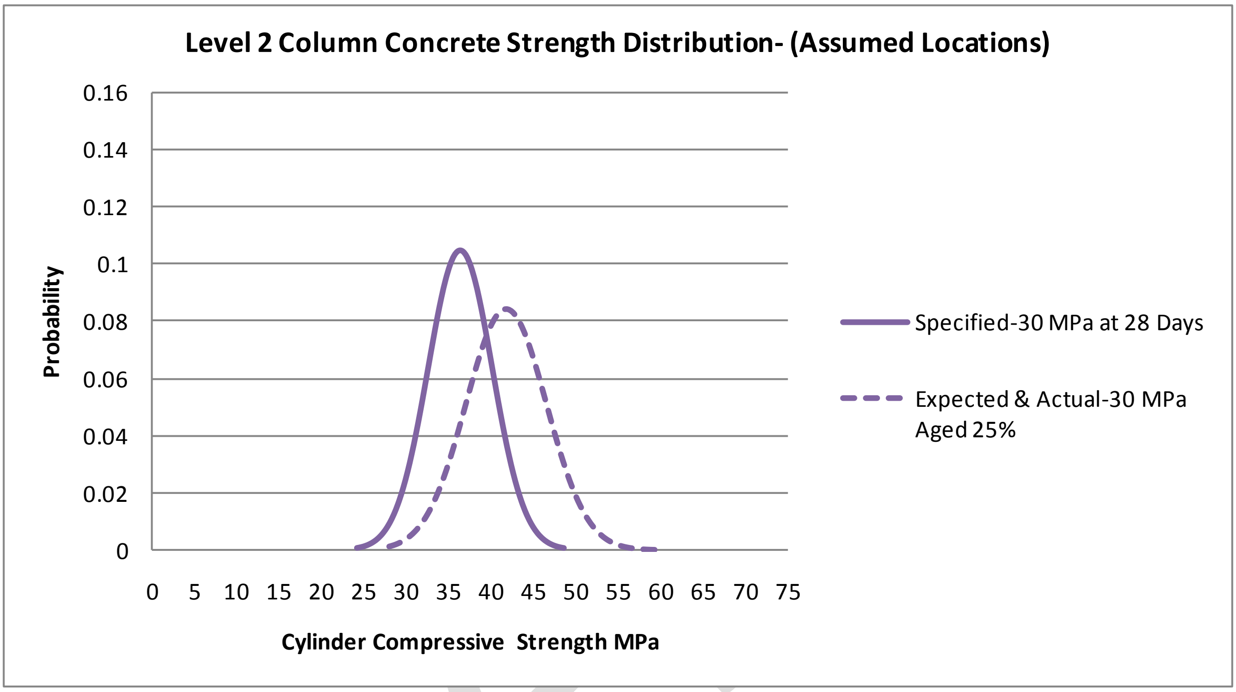
Figure 44 Level 3 to 6 Concrete Column Properties (From Top) a) Specific Level 5 and 6 columns; b) Other Level 3, 4 and 5 columns

In conclusion, on the basis of 25% strength-aging at the time of the tests, the columns from Level 5 and 6 would not have complied with the specified requirements for concrete with a specified strength of 25 MPa at 28 days with an acceptable level of confidence. However they would have complied with requirements for concrete with specified 28 day strength of 17.5 MPa with an acceptable level of confidence (Figure 44).

All other columns tested that are likely to have come from Levels 3 to 5, on the basis of 25% strength-aging at the time of the tests, would not have complied with the specified minimum requirements of concrete with specified 28 day strength of 25 MPa at 28 days, with an acceptable level of confidence.

However they would have complied with requirements for concrete with specified 28 day strength of 17.5 MPa with an acceptable level of confidence (Figure 44).

At some level, possibly Level 2, the concrete in the columns may have complied with the specified requirements of concrete with specified 28 day strength of 30 MPa at 28 days aged by 25% based on cores in C4, R6 and R7 and hammer tests of R10 (Figure 45).



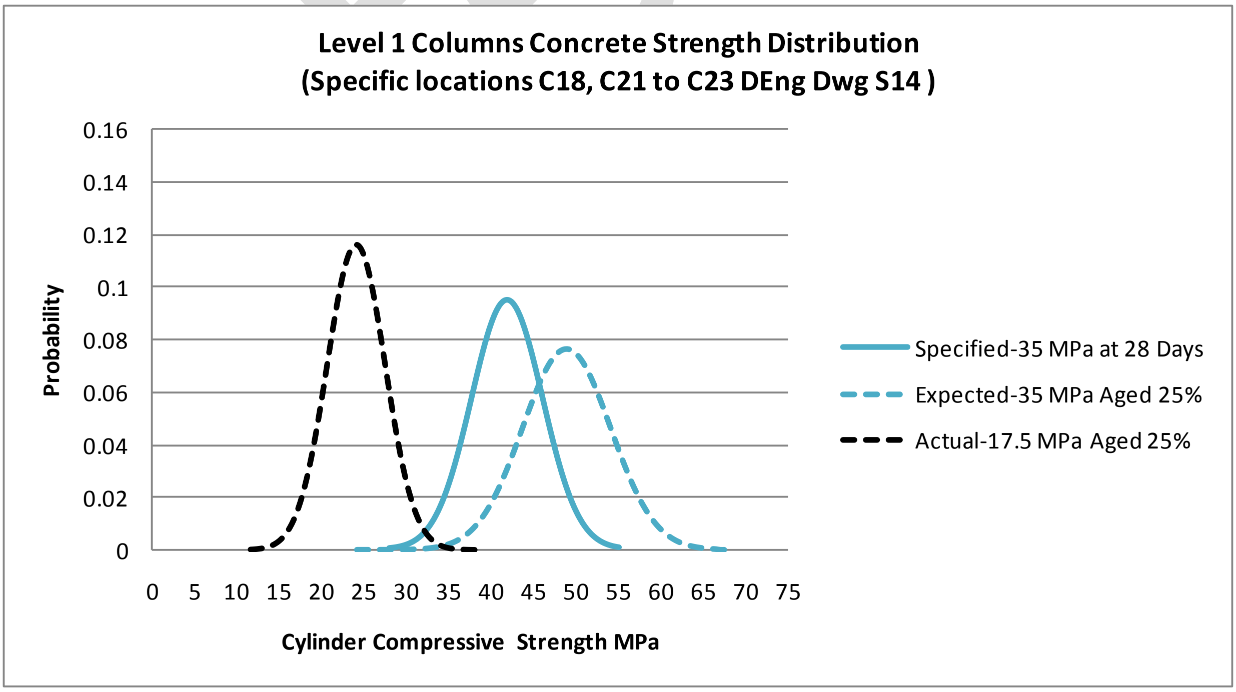


Figure 45 Level 1 and 2 Concrete Column Properties (From Top) a) Assumed Level 2 columns; b) Specific Level 1 columns

g. Discussion of Concrete Column Properties

The DENG Specification required the concrete strength to be 30 MPa at 28 days at Level 2 and 25 MPa at 28 days at Level 3 and above. The tests on Columns R6 and R7 show that the concrete changed from 30 MPa to 17.5 MPa at one level when allowance is made for normal aging. That level is likely to have been Level 3.

In summary at the time of collapse the columns from Level 3 and above are considered to have had concrete with the distribution of properties for specified 28 day strength of 17.5 MPa aged by 25% (Figure 44).

For Level 1 and Level 2 columns there remains uncertainty as to the consistency of the strengths.

The Burwood tests showed some concrete with properties consistent with 28 day strength of 30 MPa aged by 25% (Figure 45).

However core tests on the 400 mm square column stub at the CTV site on Line 4 adjacent to the lift core (C18) (Figure 40), found low quality concrete with strengths not achieving that of concrete with 28 day strength of 17.5 MPa and aged 25%, when 35 MPa at 28 days had been specified (Figure 45).

The single level columns at the entry at Grid F/ 4 were also found to only conform to the requirements for concrete with 28 day strength of 17.5 MPa, where 25 MPa concrete had been specified. However these were cast after the main structure had been built.

It is therefore considered that some of the concrete columns at Level 1 and 2 had properties consistent with the DENG Specification of 35 and 30 MPa at 28 days respectively and aged by 25% and some consistent or similar to that with 17.5 MPa at 28 days and aged by 25%.

Figure 46 400 mm Diameter Columns at Burwood showing similar base and/or mid-height failures (Left to right, top row down) a) Test item C6 Level 6 column head with mid-height failure; b)C7 with base and mid-height failures; c) C8 with base flexural (near end) and mid-height spearhead failures; d) C9 Level 6 column head with mid-height failure; e) C11 base (near end)and mid-height failure; f) C12 base and mid-height failure; g) C13 base and mid-height flexural failure(near end); h) C3 similar to C13 but lower bars have been cut during de-construction at start of spalling.

Figure 47 Level 1 Entry 400 mm Diameter Columns (left to right) a) Test item C2, 6-D12 vertical bars fractured at base (DENG Dwg S14 C23); b) Item C10 with down pipe cast in (DENG Dwg S14 C21 or C22)

Figure 48 400 x 300 mm Rectangular Columns (Left to right, top down) a) R1 beam-column joint failure at base, mid-height failure; b) Level 6 to Roof base or beam-column joint failure; c) R 3 failure base and top; d) R4 Level 6 to Roof with beam-column joint failure, still connected by rebar to e) R4' below which also f) indicates beam-column joint failure at R4' base (near camera); g) R8 with damage from mid-height still connected to R9 above h) with beam-column joint failure with i) underside of R9 smooth.

Figure 49 400 x 300 mm Rectangular Columns (from left to right and top down) a) R6 base at far end connected by reinforcing to b) R7 below, with beam-column joint failure; c) R10 remnant; d) R10' remnant.

9. Conclusions

The site examination and materials testing have resulted in the following conclusions.

1. Concrete strengths were found consistently to be lower than acceptable confidence limits for what would have been expected for concrete that had originally complied with the specification during construction.

2. The reinforcing steel was found to have properties consistent with the standards of the time.

3. A portion of reinforcing steel removed from the Line 1 shear wall near ground level was found to have work hardened during the earthquake and prior to the collapse of the building.

4. 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.

5. A northward lean on the Line 4 and 5 lift and stairwell core was found that was greater that was concluded to have occurred during construction.

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

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

8. Connection of the slabs by reinforcing steel into the Line 4 lift core walls was non-existence in some cases at Level 2, 3 and 4. Steel drag bars had been added some time after initial construction at levels 4, 5 and 6 and were not shown on the Building Consent drawings.

9. The connection of the C18 column into the lift core wall at Level 7 was less than specified and the bars had de-bonded.

10. A number of circular columns examined showed mid-height hinging as well as hinging at the base. This was seen in one column identified as being a perimeter column located between precast spandrel panels. Other circular columns were full height with hinging damage top and bottom.

11. Rectangular columns typically showed beam column joint failure where the beam had pulled out, as well as other forms of damage.

10. References

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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 50 and Figure 51).

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)

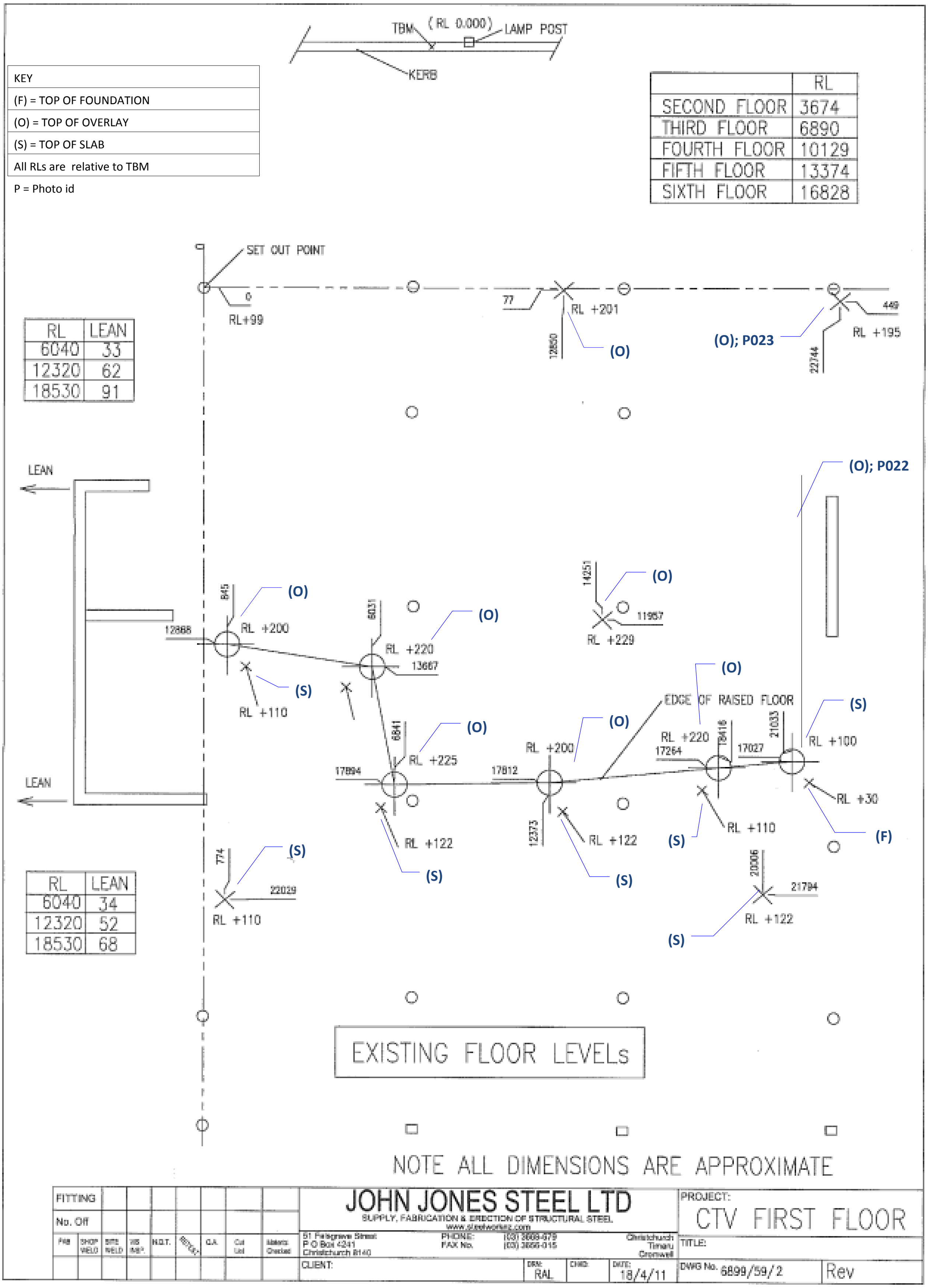

Figure 50 JJS Dwg 2: Location of overlay edge and lift core lean annotated with levels on adjacent concrete identified as 100mm overlay (O), 125mm slab (S) or foundation beam (F).

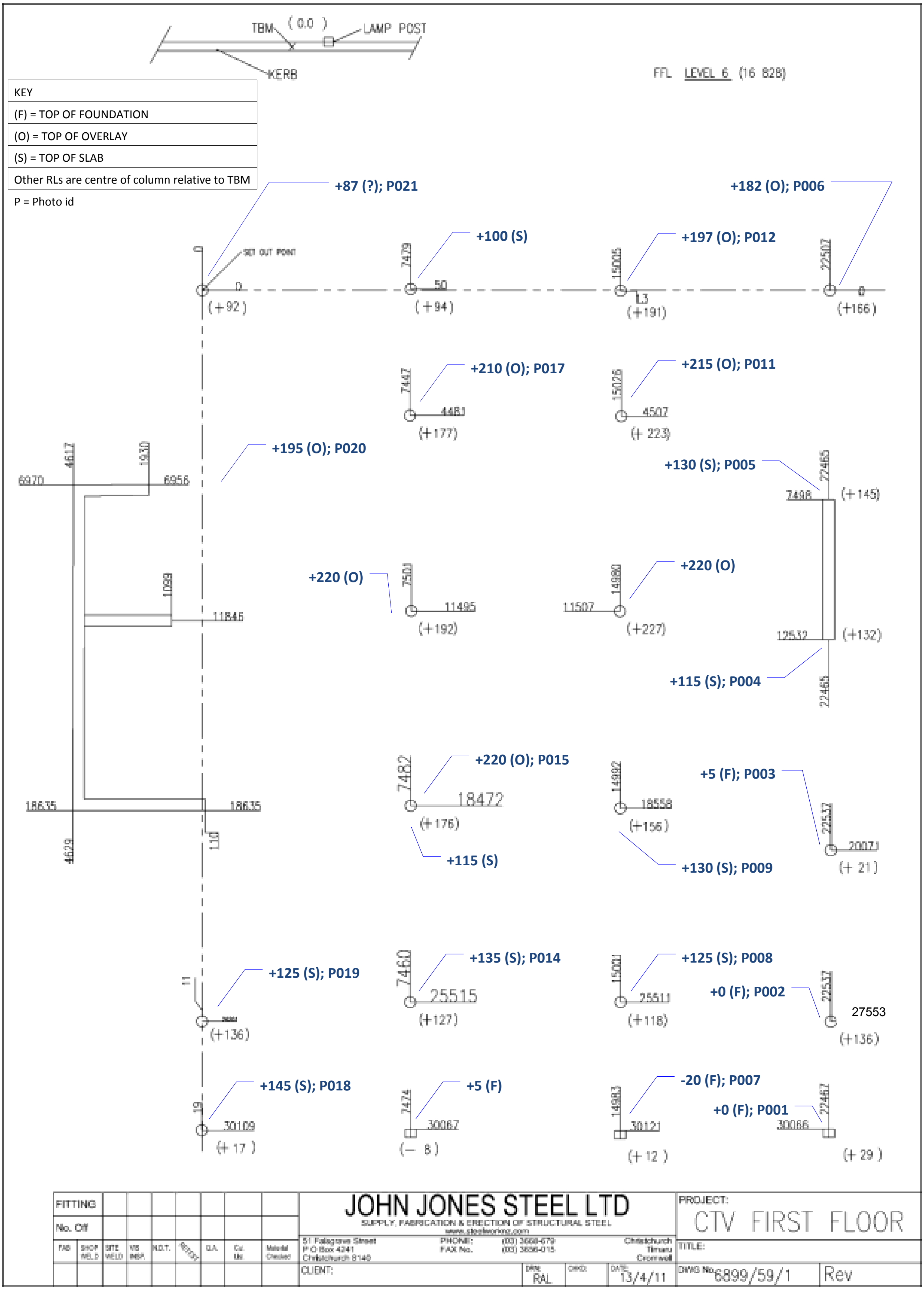

Figure 51 JJS Dwg 1: Locations and levels at centres of demolished columns, annotated with levels on adjacent concrete identified as 100mm overlay (O), 125mm slab (S) or foundation beam (F).Photo locations are designated P###.


P001


P002


P003


P004


P005


P006


P007


P008


P009


P010


P011


P012


P013


P014


P015


P016


P017


P018


P019


P020


P021


P022


PO23

Appendix B: Foundation Inspection

The following are notes for the photos during the inspection by a CERA engineer in Figure 53. The location at which the photos were taken is shown in Figure 52.

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.

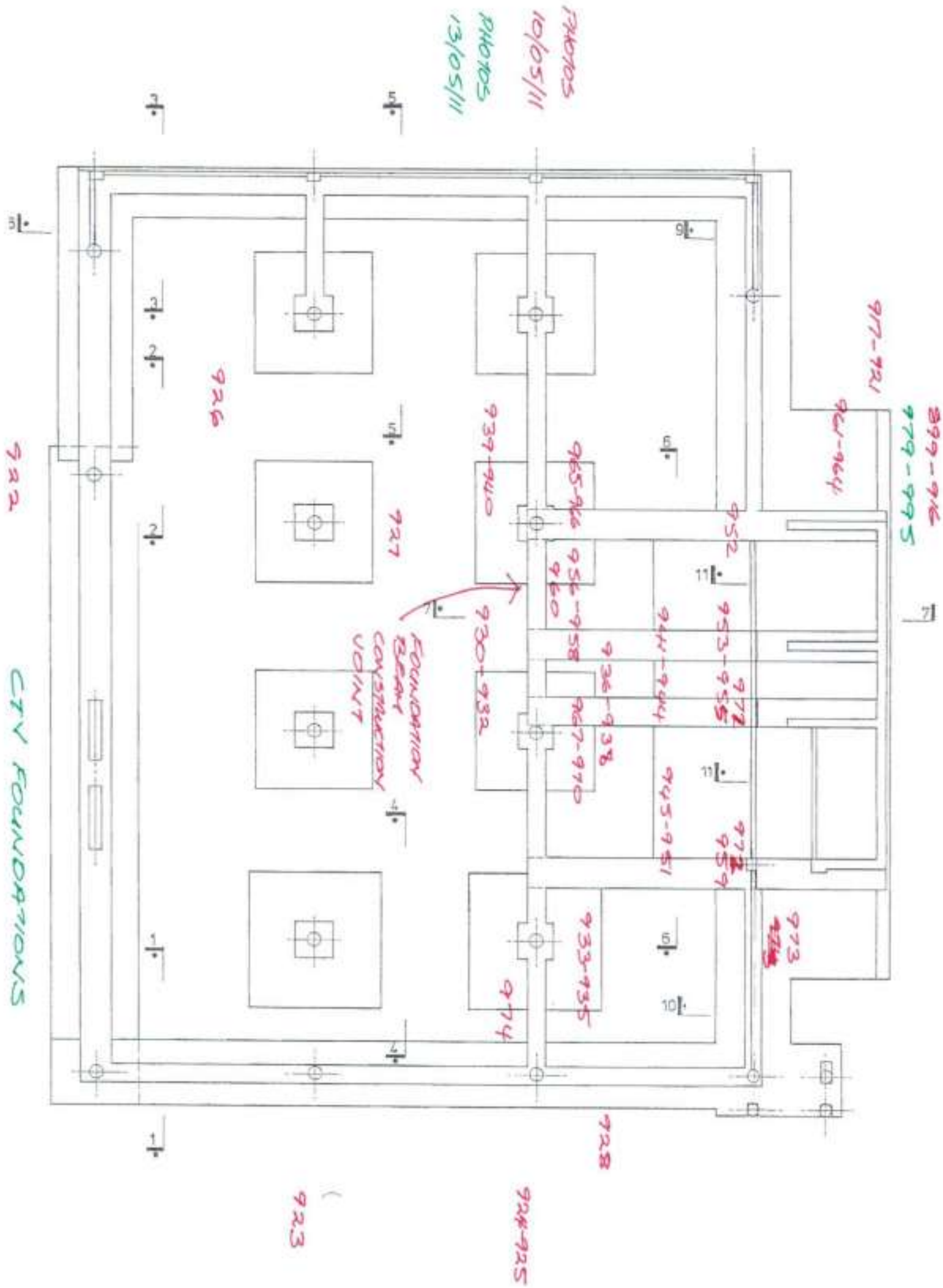

Figure 52 Locations of photos taken during foundation inspections on 10th and 13th May, 2011 (CERA)

Figure 53 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 Using Rebound Hammer

a. 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
C12	Top	42.1	1200			42.1	26.5	16	27.5	27.0	29.8
C4	Top	49.9	1200	45.9	1100	47.9	47.8	45.3		46.6	41.0
C12	Top	46.1	1200	42.0	1200	44.1	27.1	26.2		26.7	33.3
R3	Top	35.4	1200	35.3	1200	35.4	20.5	20.1		20.3	20.6
R6	Top	37.8	1200	35.7	1200	36.8	24.5	26.4		25.5	22.2
R7	Top	46.5	1200	46.2	1200	46.4	39.5	42.2		40.9	37.8

Note: C12 core 2 has been excluded as an outlier for developing strength vs hammer number relationship

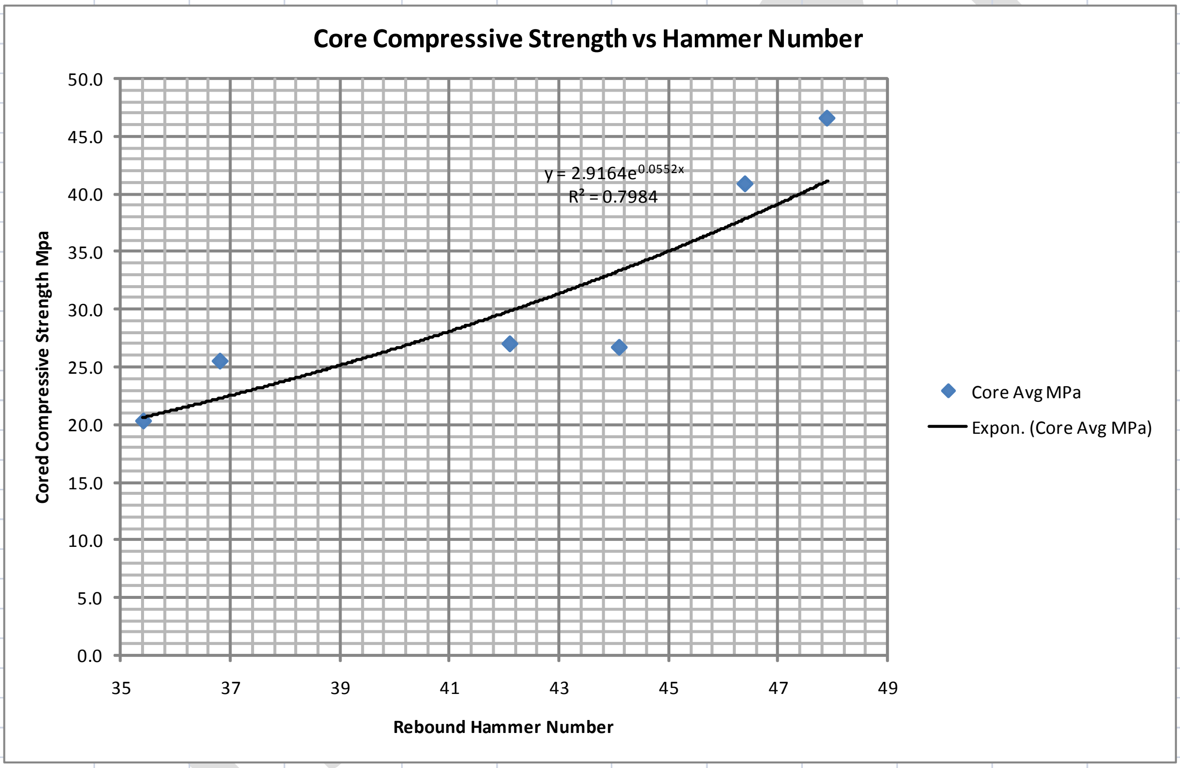

Figure 54 Strength vs Hammer Number relationship derived for columns cored and hammered per ASTM C805

b. 400 mm Diameter Columns Test Data and Observations

CTV Columns Inferred Strengths from 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	Inferred Strength from Hammer MPa	Inferred Specified 28 day Strength (5%)	Location in Building	Comments
400 mm Diameter Columns														
C1	Bottom	40.3	1200			40					27.0	20.0	Level 6 to Roof	E25; Flexure base
C1	Middle	40.8	1200			41					27.7	20.0	Connected to C8 below	
C1	Top	42.1	1200			42	26.5	16.0	27.5	23.3	29.8	20.0		
C2	Bottom	35.9	1200			36					21.2	17.5	Level 1 to 2 C23	6 D12 bars fractured
C2	Top	48.1	1130			48					41.5	30.0		
C3		41.6	1130			42					29.0	20.0	Level unknown	Flexure base and mid-height
C4	Bottom	49.3	1100			49	47.8	45.3		46.6	44.3	35.0	Level unknown	Flexure base and top
C4	Top	45.9	1100	49.9	1200	48					41.0	30.0		
C5	Bottom	41.1	1200			41					28.2	20.0	Level 6 to Roof	Flexure base and mid-height
C5	Top	38.3	1130			38					24.2	17.5		
C6		35.6	1130			36					20.8	15.0	Level 6 to Roof Edge	Flexure mid-height
C7		41.9	1200			42					29.5	20.0	Level unknown; Edge	Flexure base and mid-height
C8		33.5	1130			34					18.5	15.0	Edge; Level 5 to 6; C1 above	Flexure base and mid-height
C9	Top	37.6	1200			38					23.2	17.5	Level 6 to Roof Edge	Flexure base and mid-height
C10	Bottom	35.4	1200			35					20.6	15.0	Level 1 to 2 C21 or C22	6 D12 bars fractured
C10	Top	31.1	1200			31					16.2	12.5		Downpipe in column
C11		34.3	1000			34					19.4	15.0	Level unknown; Edge	Flexure base and mid-height
C12		42	1200	46.1	1200	44	27.1	26.2		26.7	33.2	20.0	Level unknown; Edge	Flexure base and mid-height
C13		36.9	1200			37					22.4	17.5	Level unknown; Edge	Flexure base and mid-height

Table 6 400 mm Diameter Columns Rebound Hammer results, inferred strengths, locations of columns and comments on failure damage

c. 400 x 300 mm Rectangular Columns Test Data and Observations

CTV Columns Inferred Strengths from 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	Inferred Strength from Hammer MPa	Inferred Specified 28 day Strength (5%)	Location in Building	Comments
400 x 300 mm Rectangular Columns (Line A)														
R1		37.7	1130			38					23.4	17.5	Level unknown	Beam-column joint
R2	Bottom	40.4	1200			40					27.1	20.0	Level 6 to Roof	Beam-column joint
R2	Top	41.6	1200			42					29.0	20.0		
R3		35.3	1200	35.4	1200	35	20.5	20.1		20.3	20.5	15.0	Level unknown	Flexure base and top
R4	Bottom	45.4	1100			45					35.7	25.0	Level 5 to Level 6	Beam-column joint
R4	Top	46.6	1100			47					38.2	30.0	R4' above	
R4'	Bottom	39.6	1100			40					26.0	20.0	Level 6 to Roof	Beam-column joint
R4'	Top	36.4	1100			36					21.8	15.0		
R5		43.6	1130			44					32.4	25.0	Level unknown	Beam-column joint and mid-height
R6		35.7	1200	37.8	1200	37	24.5	26.4		25.5	22.2	17.5	Above R7	Beam-column joint and mid-height
R7		46.2	1200	46.5	1200	46	39.5	42.2		40.9	37.7	30.0	Below R6	Mid-height and beam-column joint
R8	Bottom	42.9	1200			43					31.1	20.0	Level unknown	Beam-column joint,
R8	Top	38.9	1200			39					25.0	20.0		mid-height flexure
R9	Bottom	39.5	1200			40					25.8	20.0	Level unknown	Beam-column joint
R9	Top	35.8	1200			36					21.0	15.0		
R10		50.1	1130			50					46.3	35.0	Level unknown	Flexure
R10'		42.9	1130			43					31.1	20.0	Level unknown	Flexure

Table 7 400 x 300 mm Rectangular Columns Rebound Hammer results, inferred strengths, locations of columns and comments on failure damage

d. CTV Columns Insitu Schmidt Hammer Impact Investigation 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

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Wigram, Christchurch 8042,
New Zealand

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CTV COLUMNS
BURWOOD LANDFILL
INSITU SCHMIDT HAMMER IMPACT INVESTIGATION

COLUMN: LOCATION:	C1 Bottom	C1 Middle	C1 Top	C2 Bottom	C2 Top	C3	C4 Bottom	C4 Top	C5 Bottom	C5 Top	C6	C7
	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

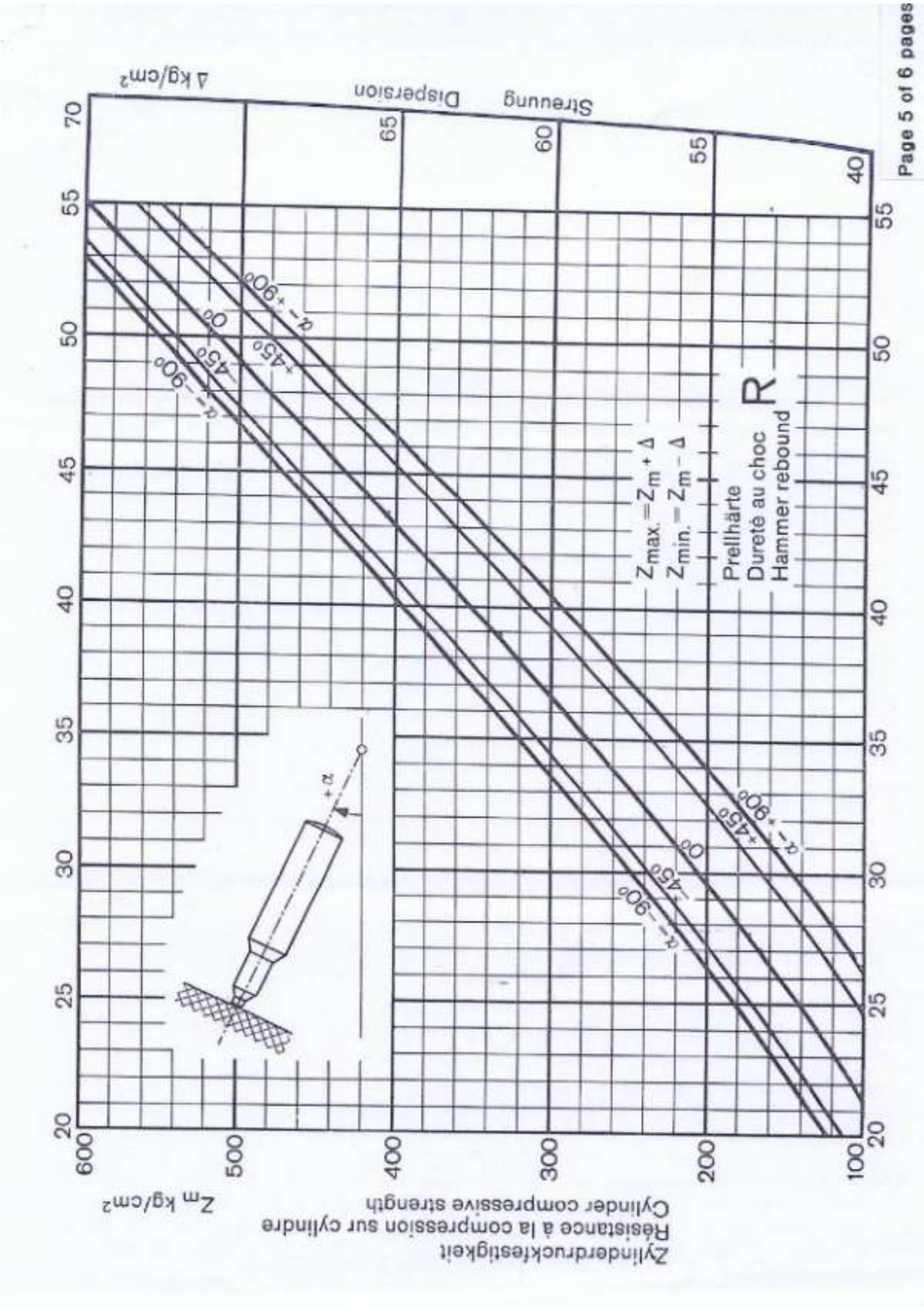
CTV COLUMNS
BURWOOD LANDFILL
INSITU SCHMIDT HAMMER IMPACT INVESTIGATION

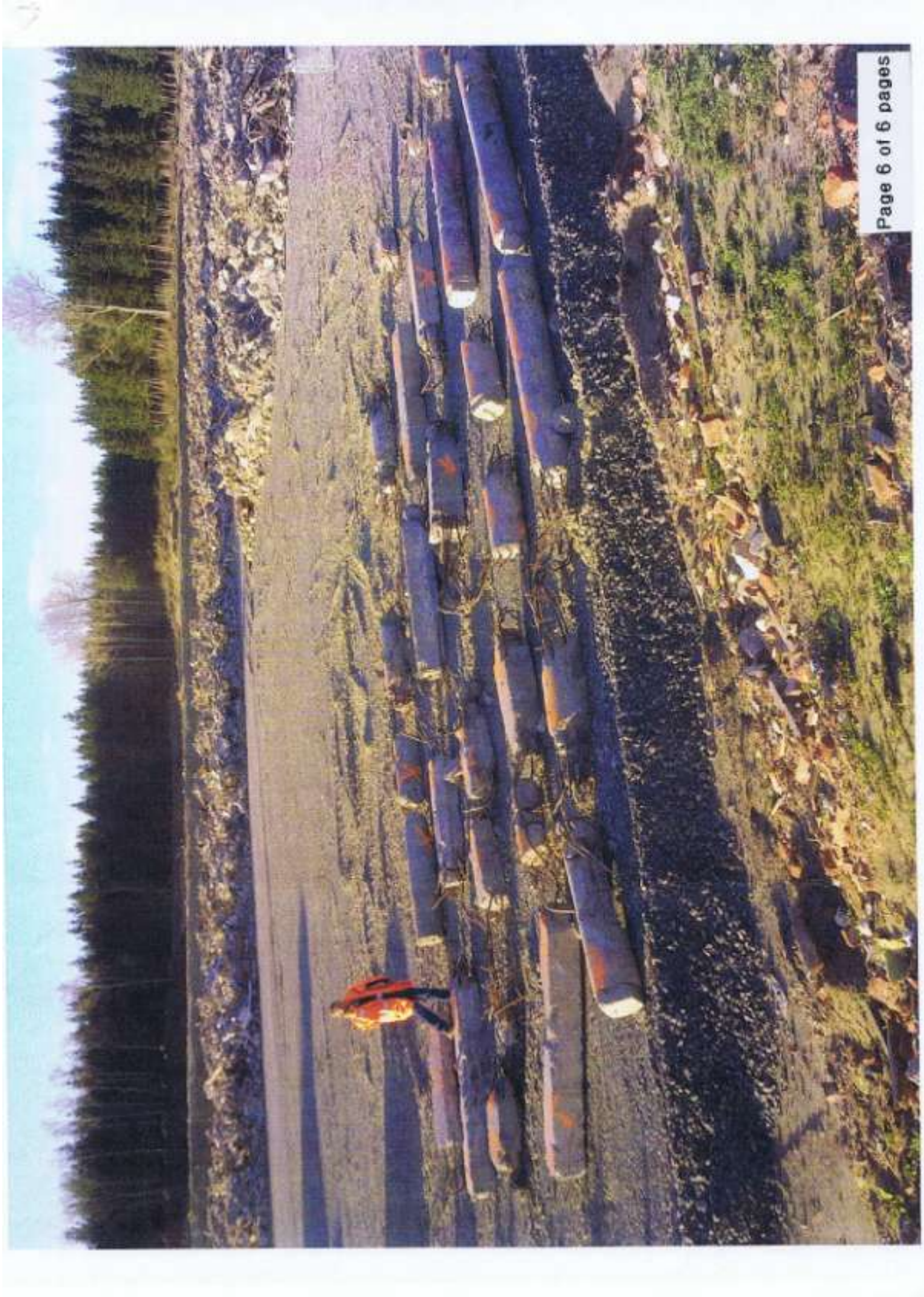
COLUMN:	R4	R4'	R4'	R5	R6	R7	R8	R8	R9	R9	R10	R10'
LOCATION:	Top	Bottom	Top				Bottom	Top	Bottom	Top		
	49	38	38	42	40	46	46	38	41	-	50	44
	49	42	40	44	-	45	46	39	36	34	51	48
	49	42	38	50	30	46	46	41	46	38	52	40
	44	38	38	41	32	42	39	36	40	37	50	41
IMPACT:	40	43	36	46	38	47	42	36	41	33	46	38
	44	42	36	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 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 4 of 6 pages


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


Page 6 of 6 pages

e. CTV Columns Schmidt Hammer Impact Recovered Cores Comparison

(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

OPUS

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

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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 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

IANZ Approved Signatory

Designation : *Laboratory Manager*
Date : 9 June 2011

Sampling is not covered by IANZ Accreditation. Results apply only to sample tested.
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PF-LAB-095 (08/12/2003)

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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 :	**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	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

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

Designation : *Laboratory Manager*
Date : 9 June 2011

laboratory

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

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

Page 2 of 2

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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 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

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

Designation : *Laboratory Manager*
Date : 9 June 2011

ia laboratory

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PF-LAB-095 (16/12/2010)

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Appendix D: Reinforcing Steel and Slab Decking Test Results

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

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

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

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

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

SAI GLOBAL (NZ) LIMITED

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Appendix E: Drilled Concrete Core Test Results

Concrete cores were cut from the line 1 shear wall element marked E4; The lower portion of the Line 5 shear wall at the stair well; in a 400 mm diameter Level 6 to Roof column marked E25; in a precast log beam and into two portions of suspended slab still attached to concrete beams.

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.

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 Shear 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 2015 (13/9/2006, modified 07/01/2008)

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b. Line 1 Level 4 Shear 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

CN 2591 (10/02/08)

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c. Line 5 Level 1 Stair Core Wall: 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 Core Wall: 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 :	**6JHFEE.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

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

CSF 2005 (15/09/2006)

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e. Level 6 400 Dia. Column E25: 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 (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

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	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 : 28 March 2011
Date reported : 29 March 2011

IANZ Approved Signatory

Designation : *Laboratory Manager*
Date : 29 March 2011

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

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f. 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

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Designation : *Laboratory Manager*
Date : 29 March 2011

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g. Precast Log Beam: Compressive Strength

**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 (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

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h. Level 1 400 Square Column C18: Compressive Strength

i. Set 1 Results

CONCRETE COMPRESSION OF CORES TEST REPORT

OPUS

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/m³)	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

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 : 5 May 2011

laboratory

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

PF-LAB-095 (16/12/2010)

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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

OPUS

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.

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Yours faithfully
Opus International Consultants

Geoff Jones
Laboratory Manager

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 :	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

OPUS

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	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 : 11 May 2011
Date reported : 11 May 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 : 11 May 2011

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

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

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CTV 1


CTV 2


CTV 3


SCORCHED
SCORCHED

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 permission of DENG and ARCH)

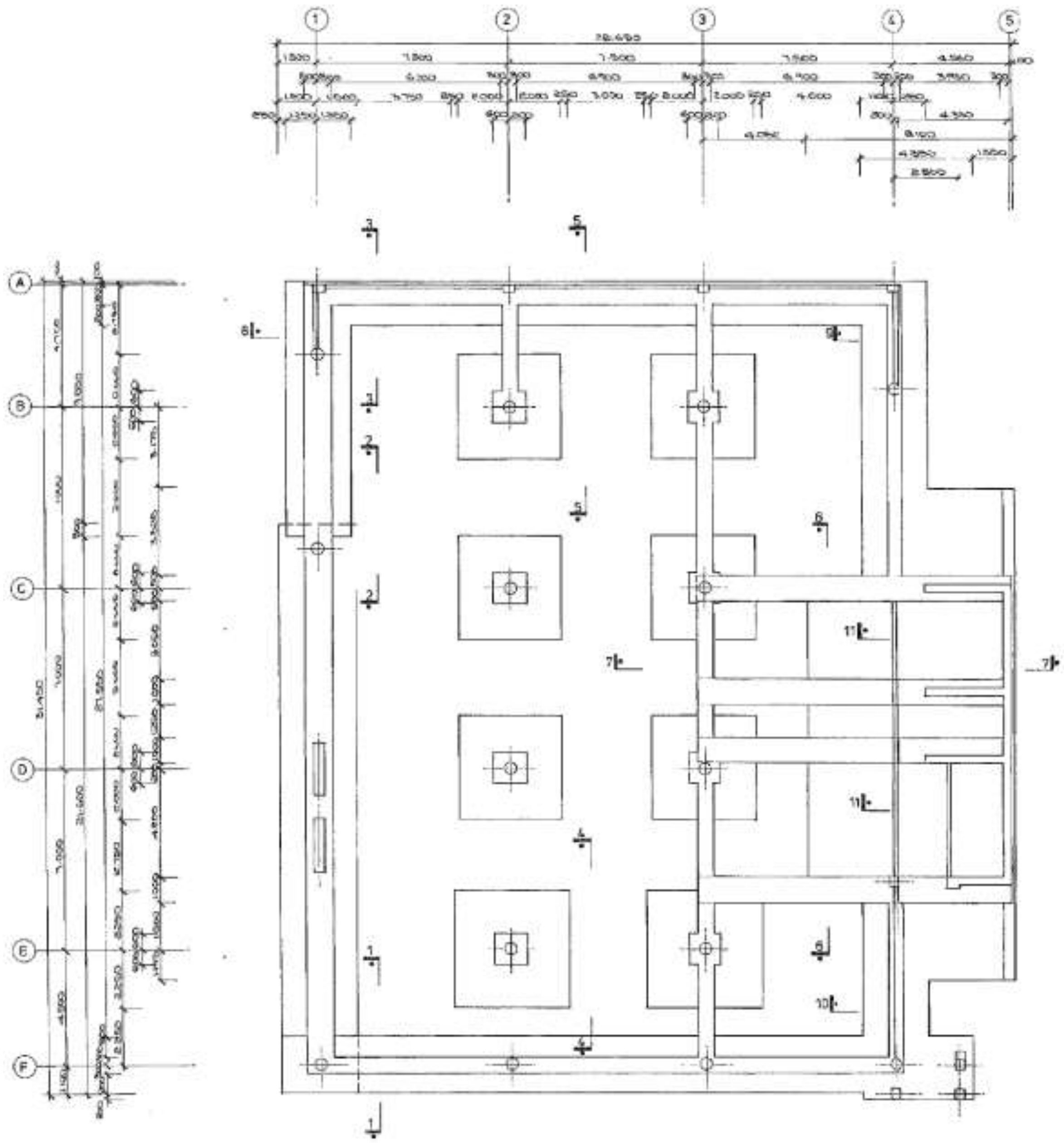

Figure 55 Foundation Layout (Extract from DENG Dwg S2)

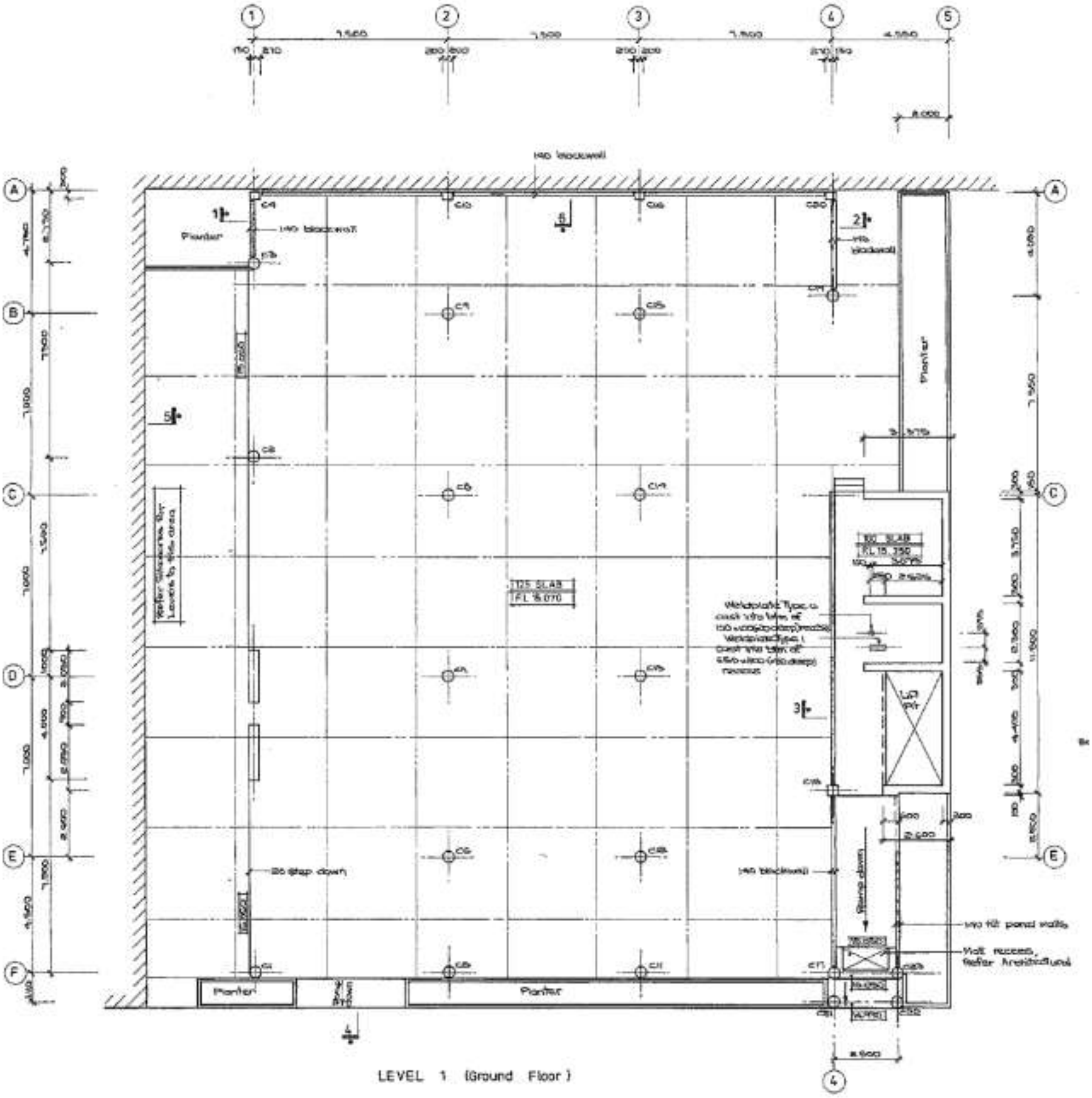

Figure 56 Level 1 ground floor slab layout (extract DENG Dwg S9)

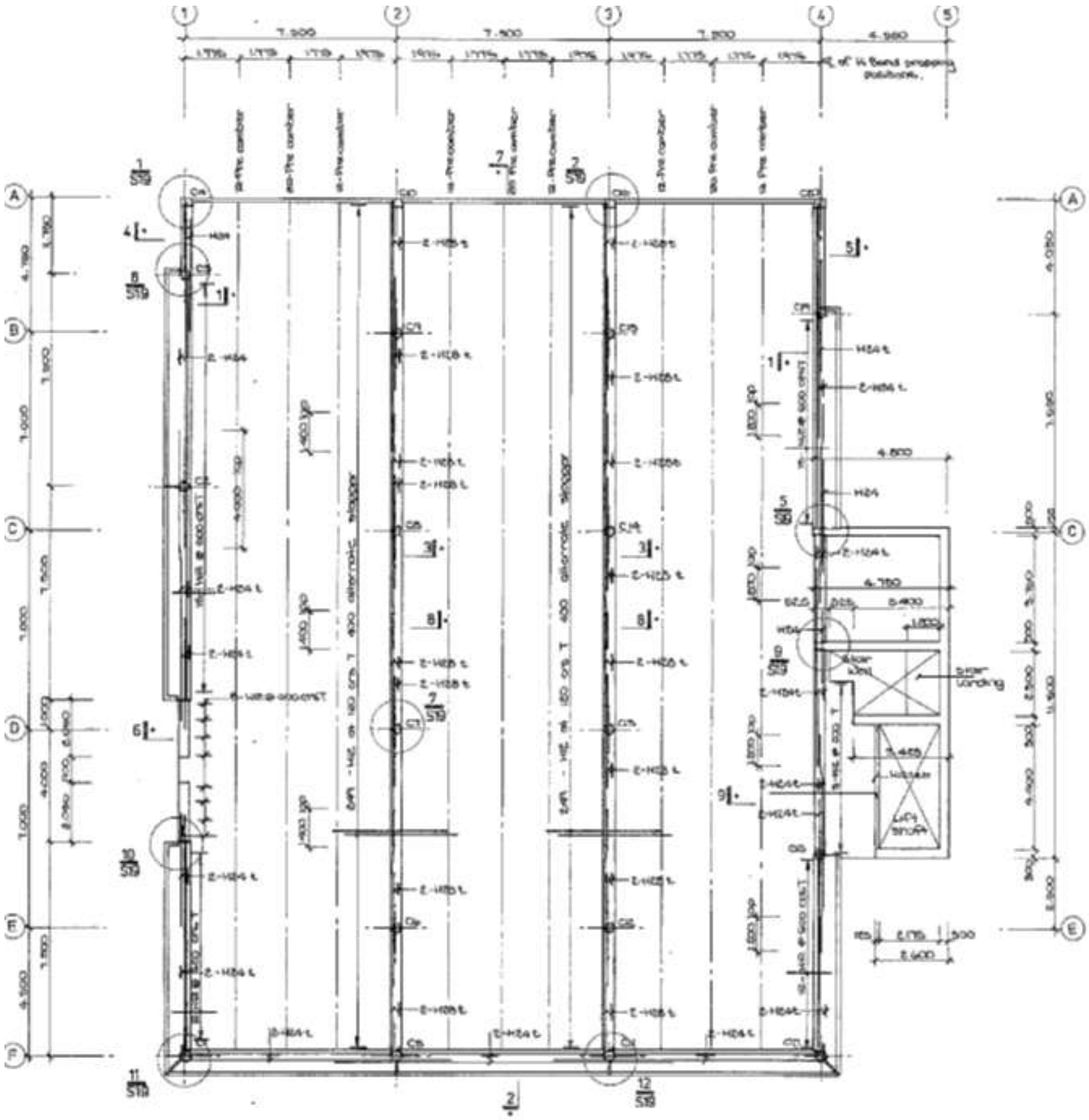

Figure 57 Level 2 to 6 Floor Layout (Extract from DENG Dwg S15)

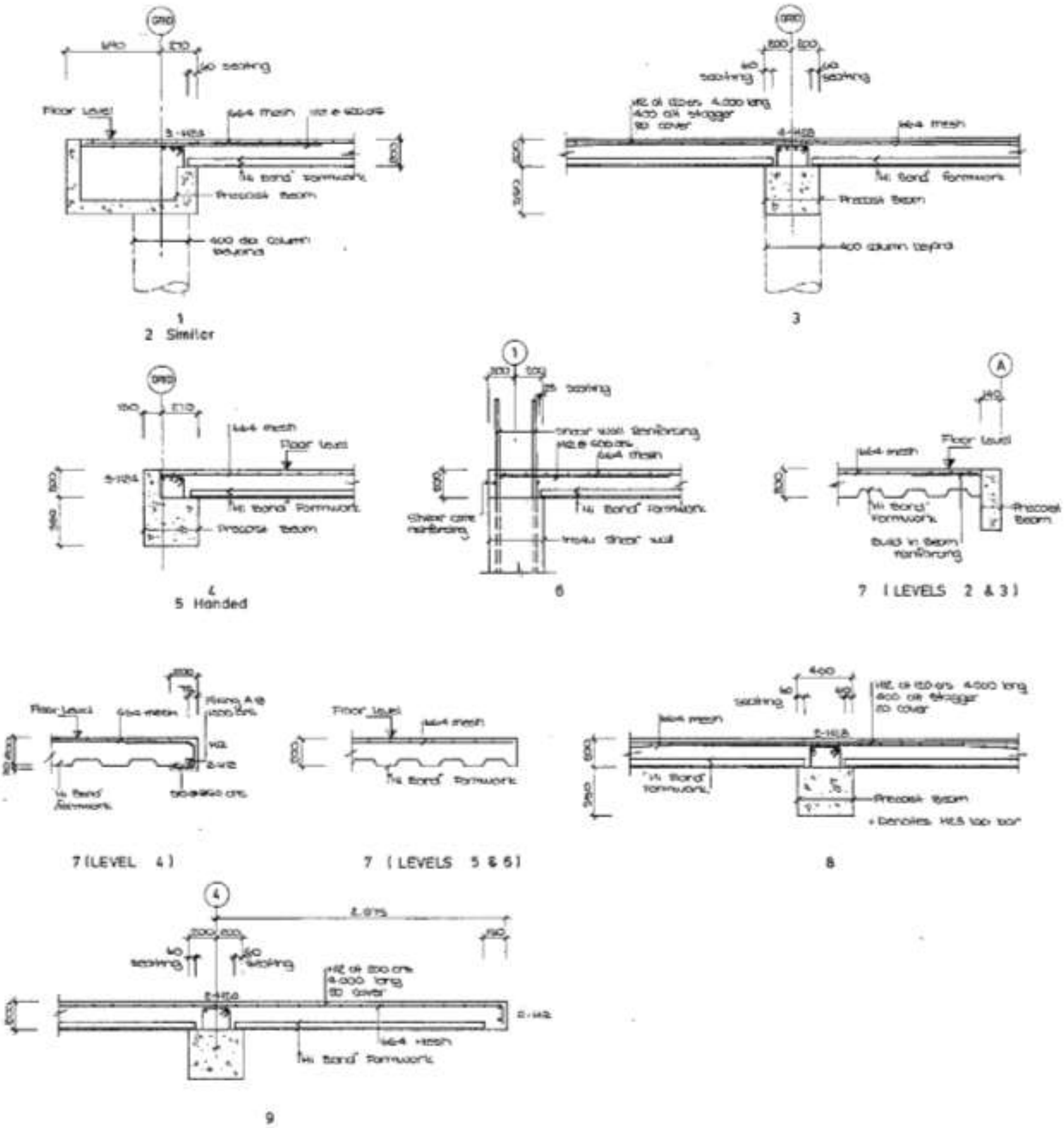

Figure 58 Level 2 to 6 floor slab details (Extract from DENG Dwg S15)

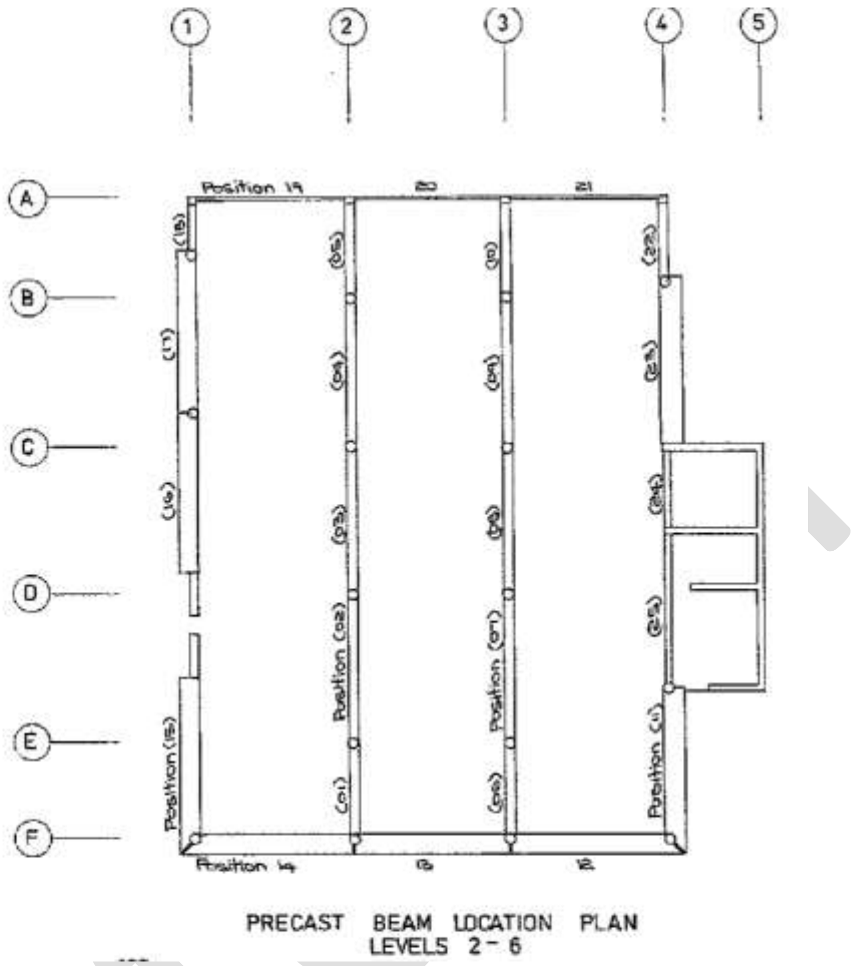

Figure 59 Precast beam layout drawings (Extract DENG Dwg S18)

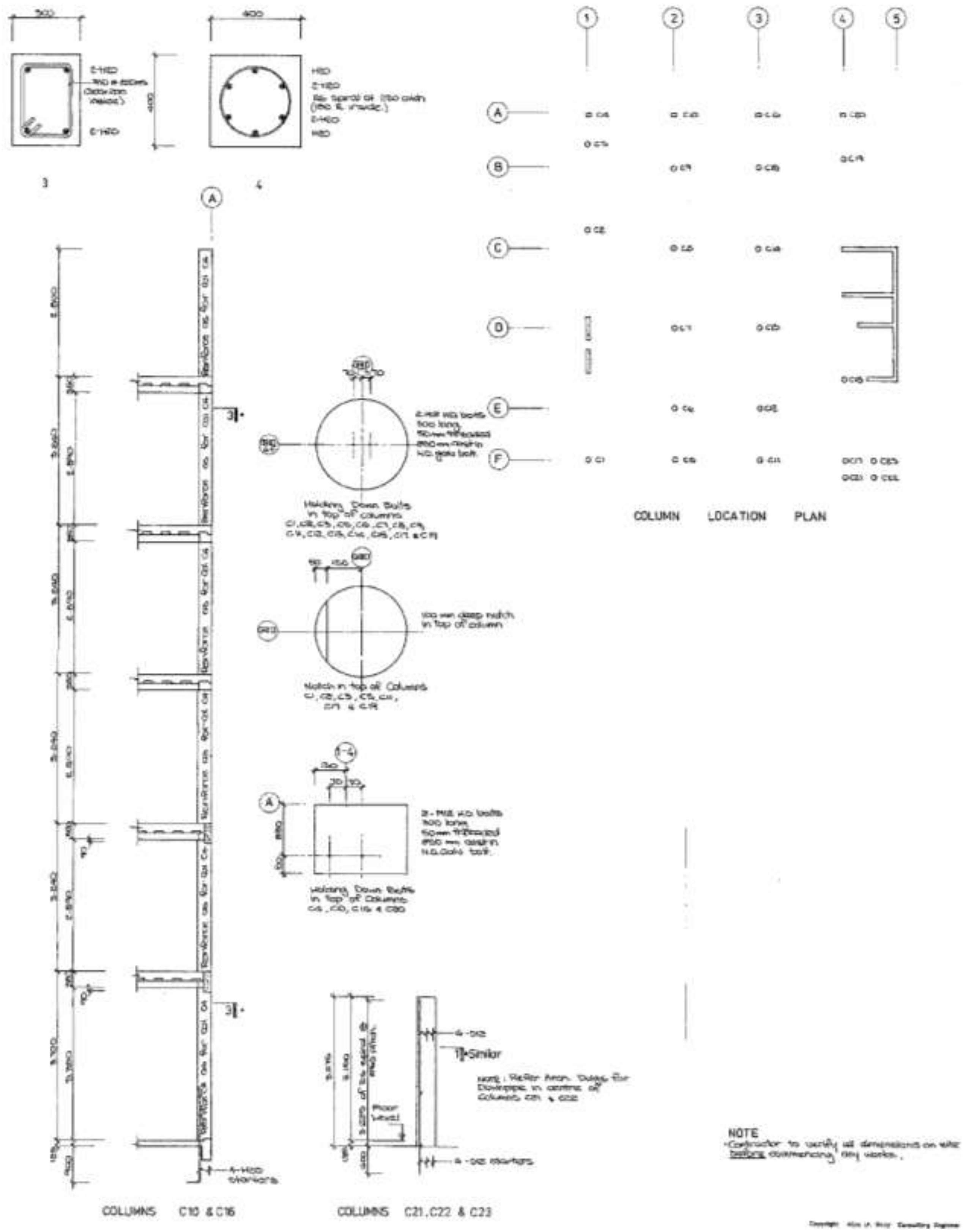

Figure 60 Columns (Extract DENG Dwg S14)

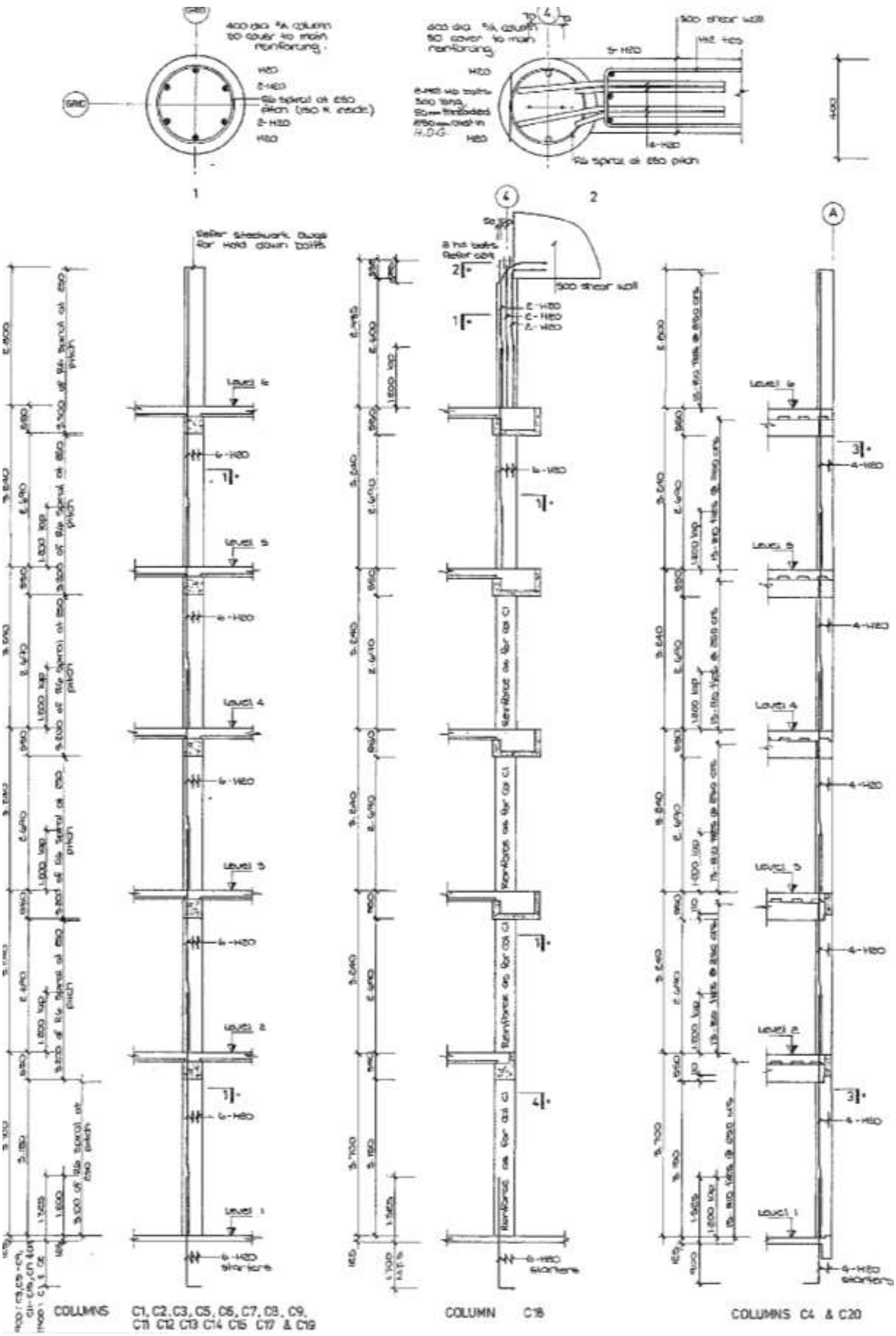

Figure 61 Columns (Extract DENG Dwg S14)

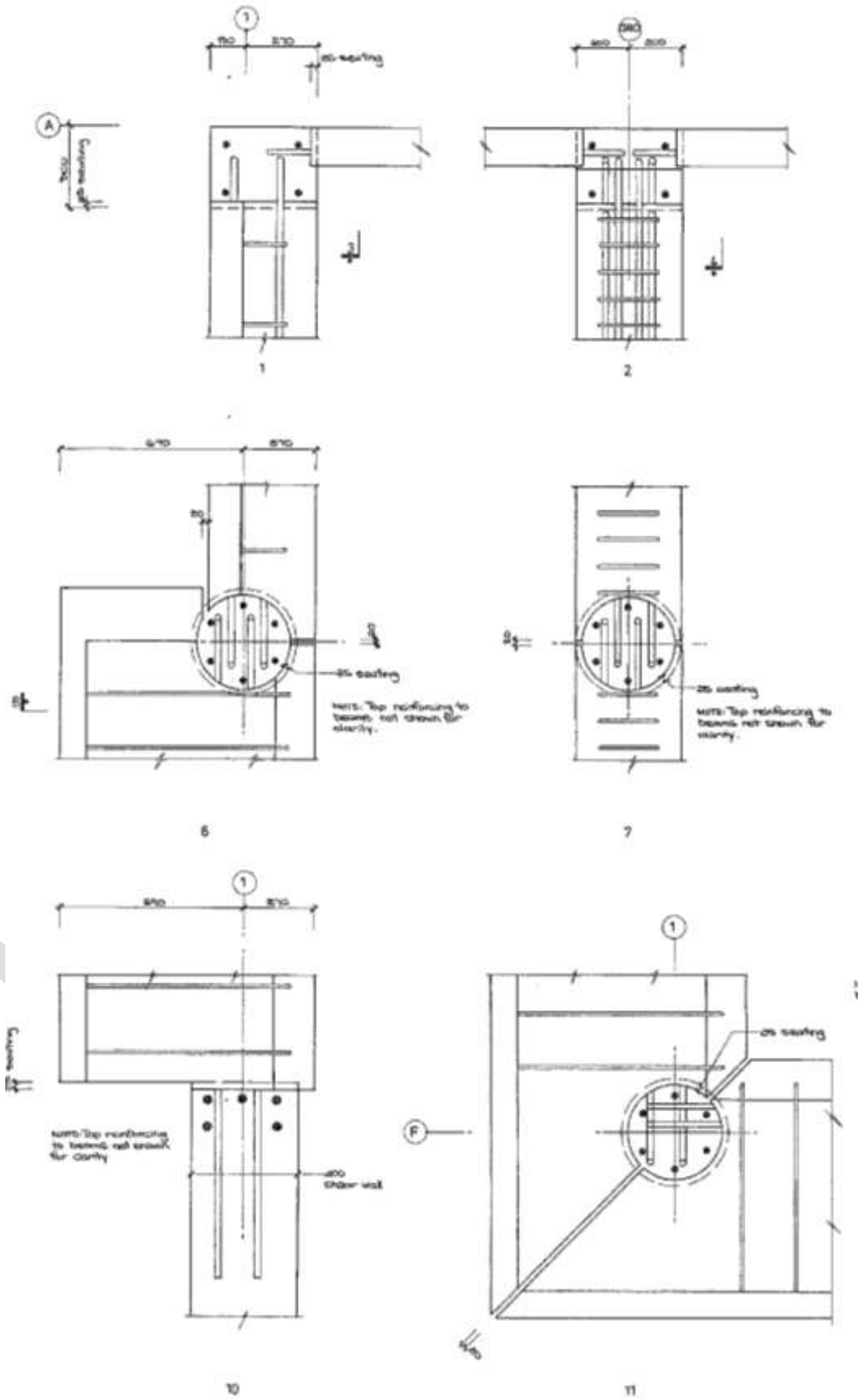

Figure 63 Beam-Column Joints (Extract DENG Dwg S19)

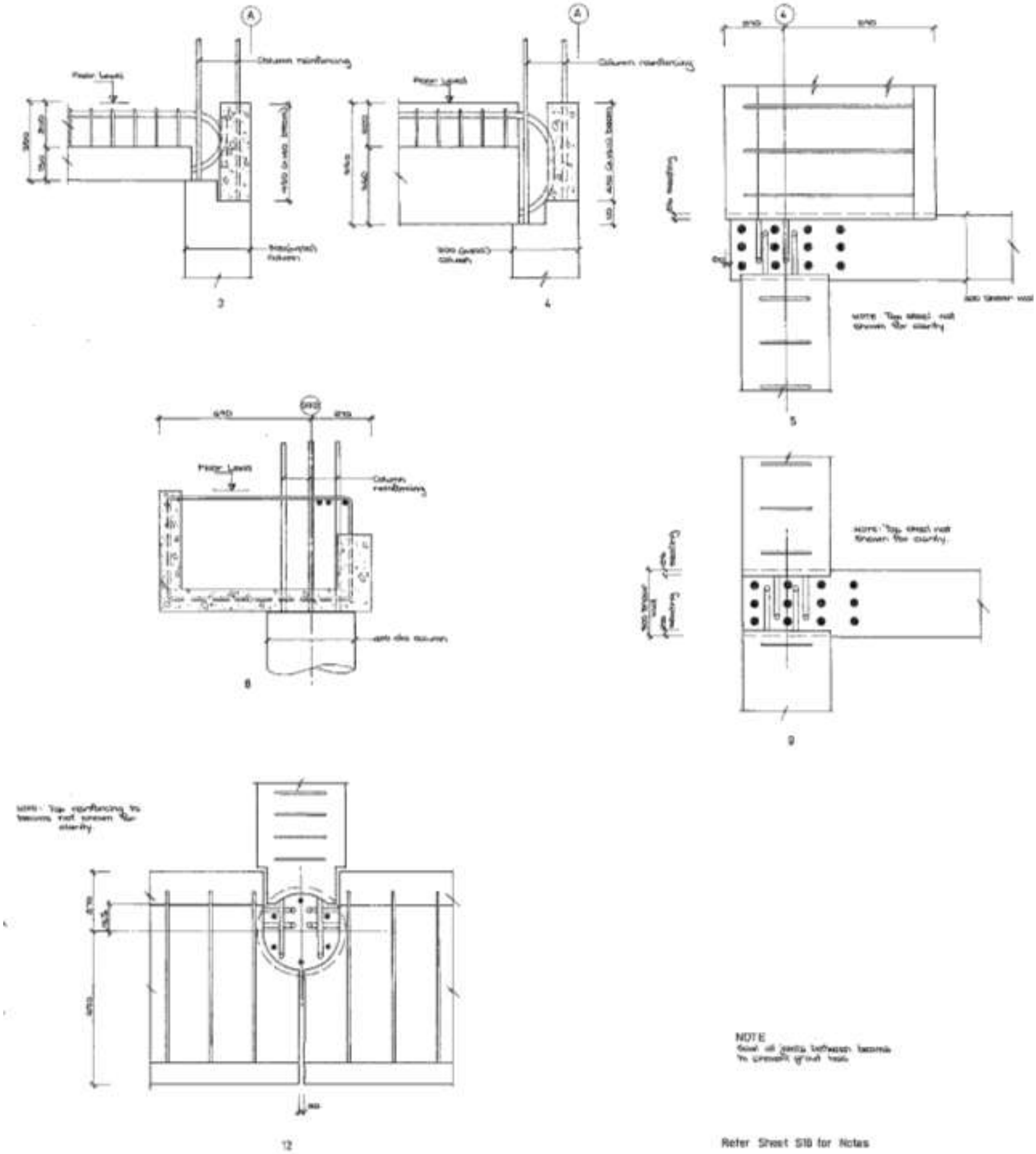

Figure 64 Beam-Column Joints (Extract DENG Dwg S19)

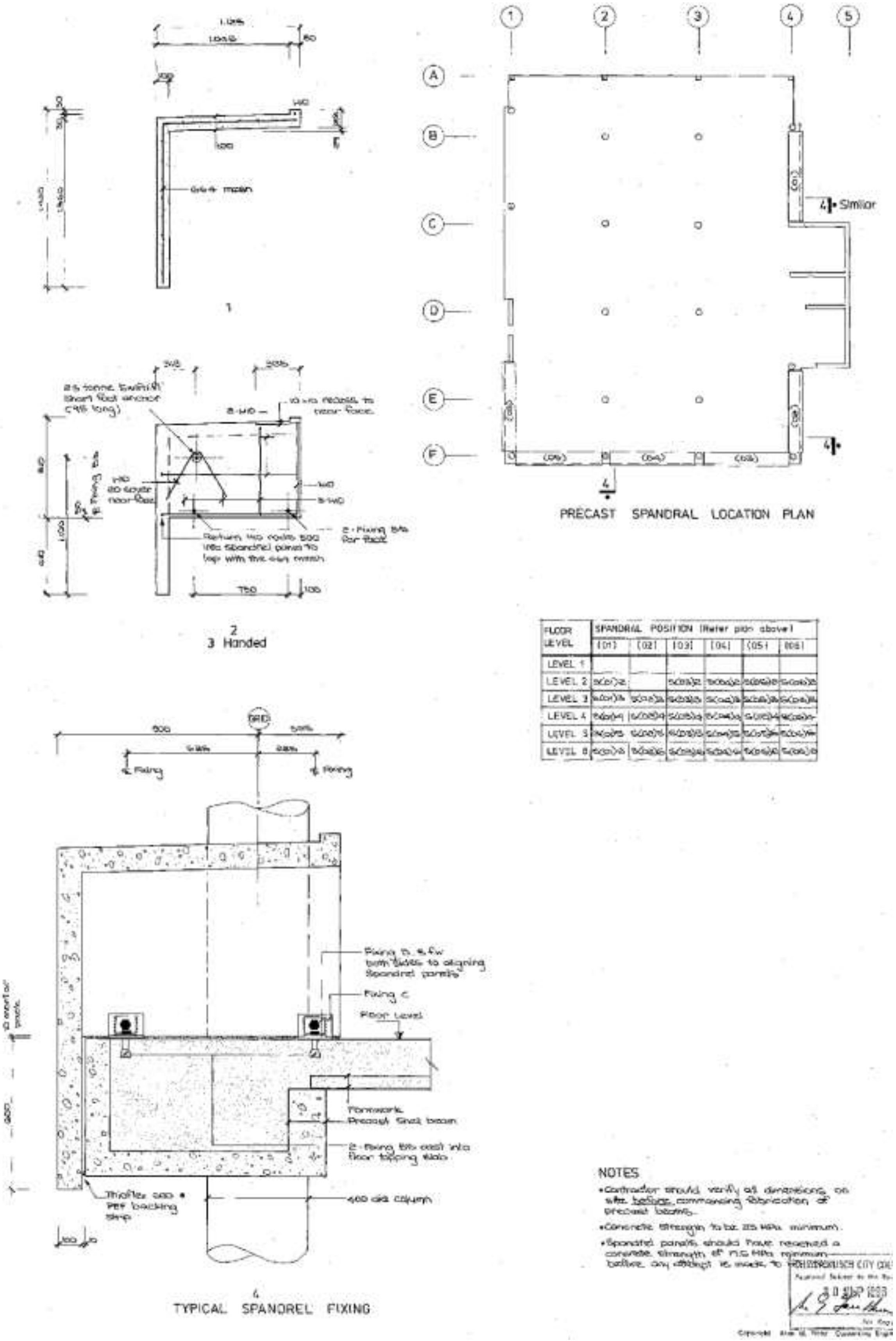

Figure 65 Pre-cast spandrel panels (Extract from DENG Drawing S25)

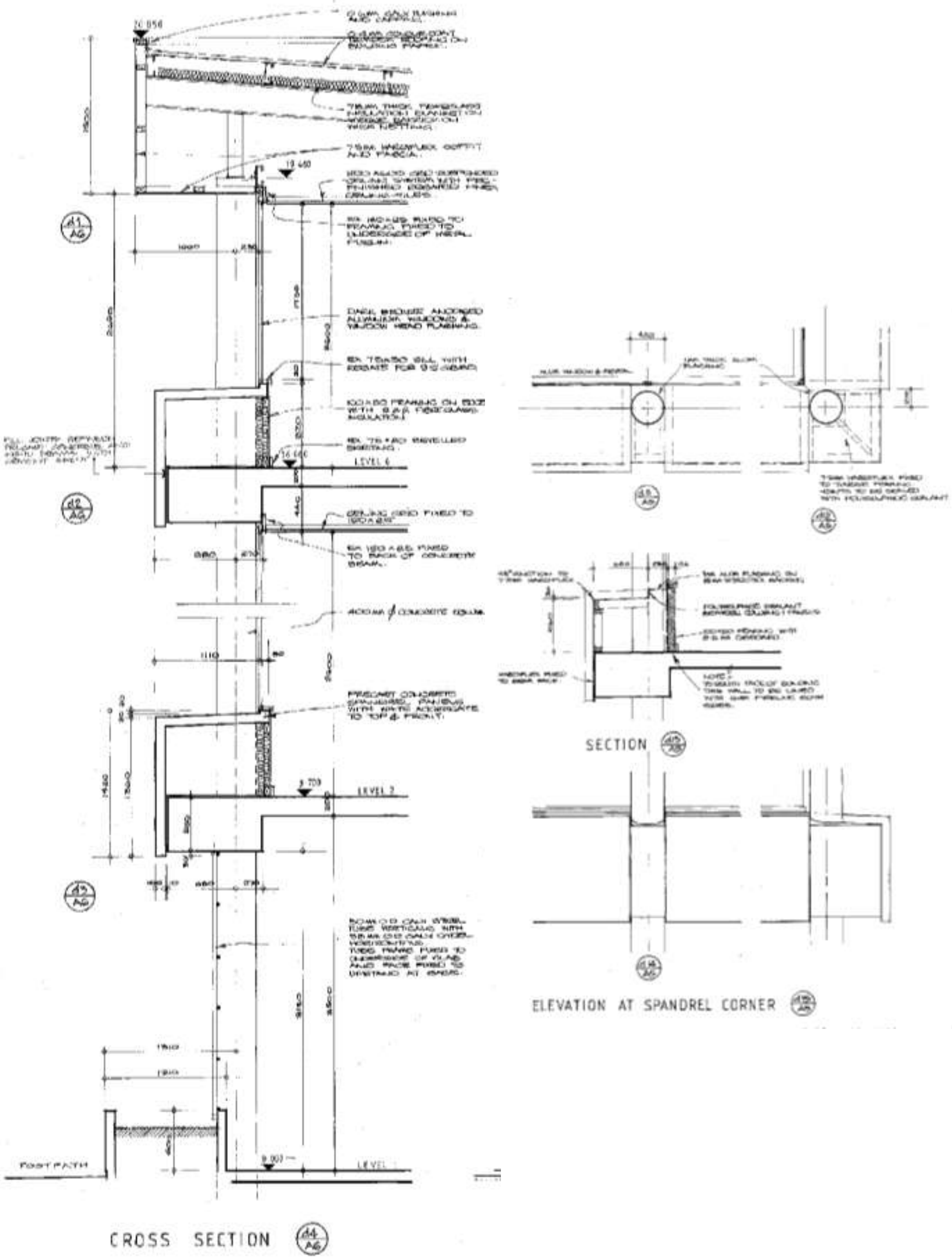

Figure 66 Spandrel Panel Details at 400 mm Diameter Columns (Extract ARCH Dwg A7)

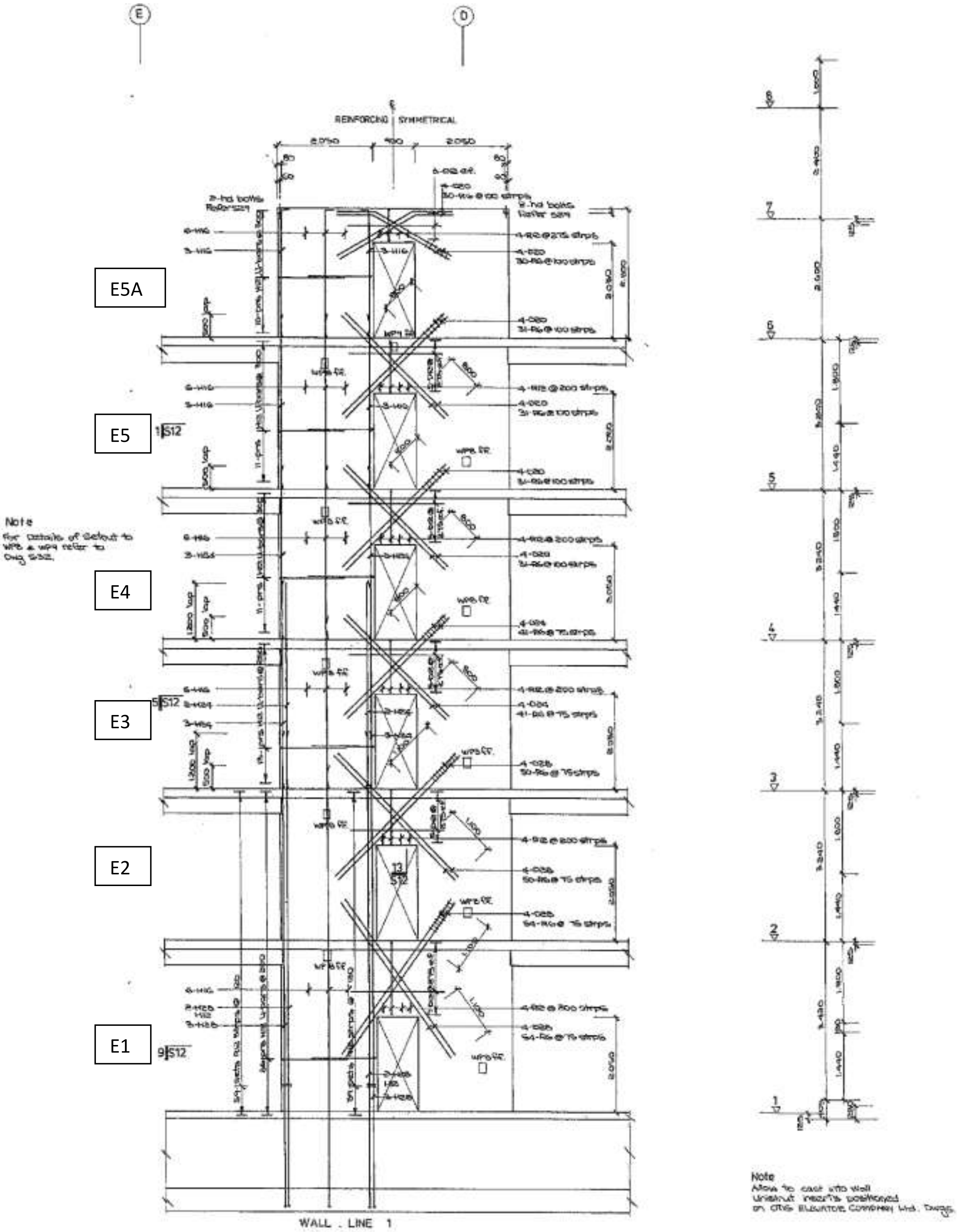

Figure 67 Line 1 Shear Wall with Items E1 to E5A identified (Extract from DENG Dwg S10)

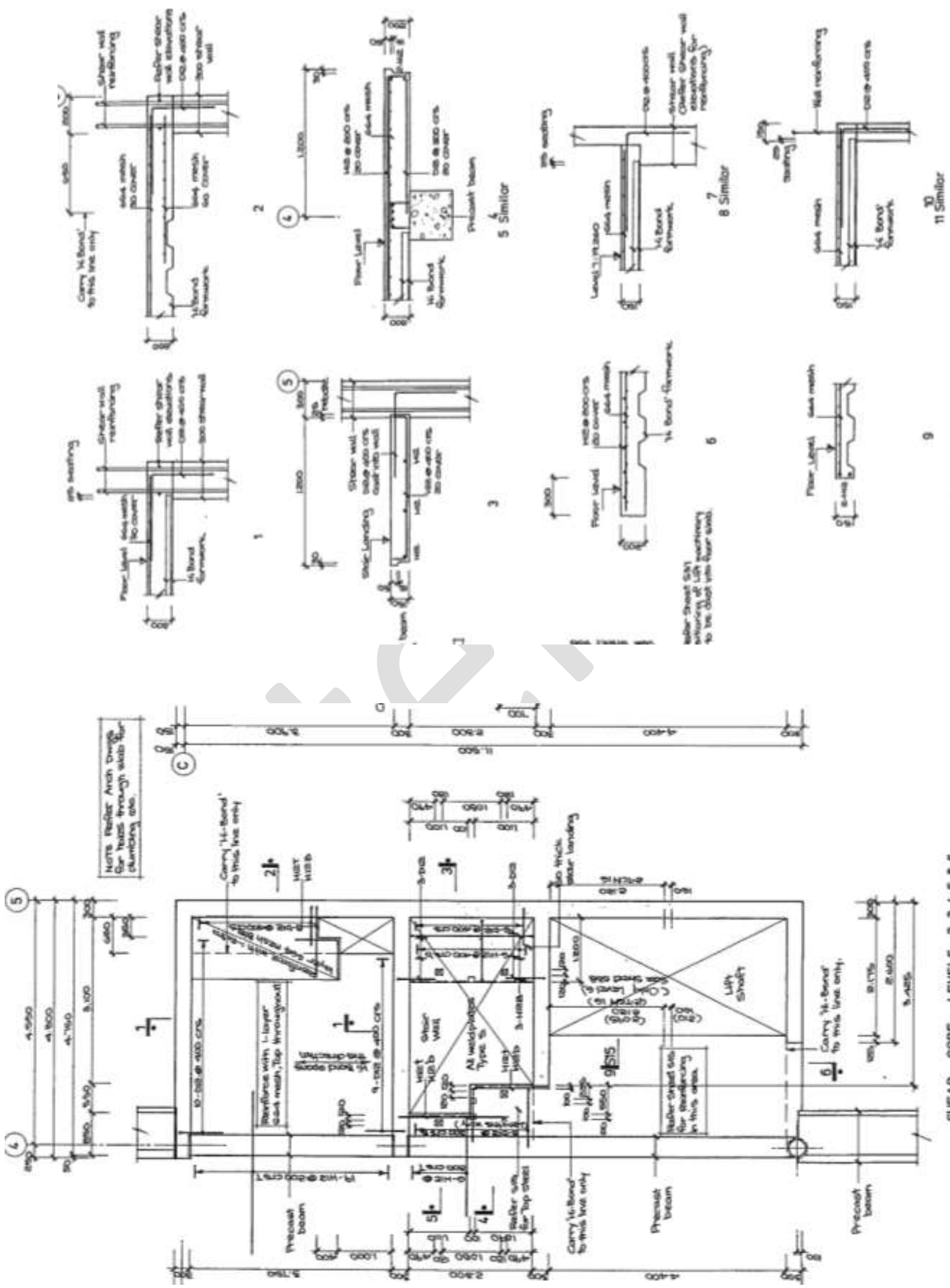

Figure 68 Shear Core Floors Level 2 to 6 and details (Extract from DENG Dwg S16)

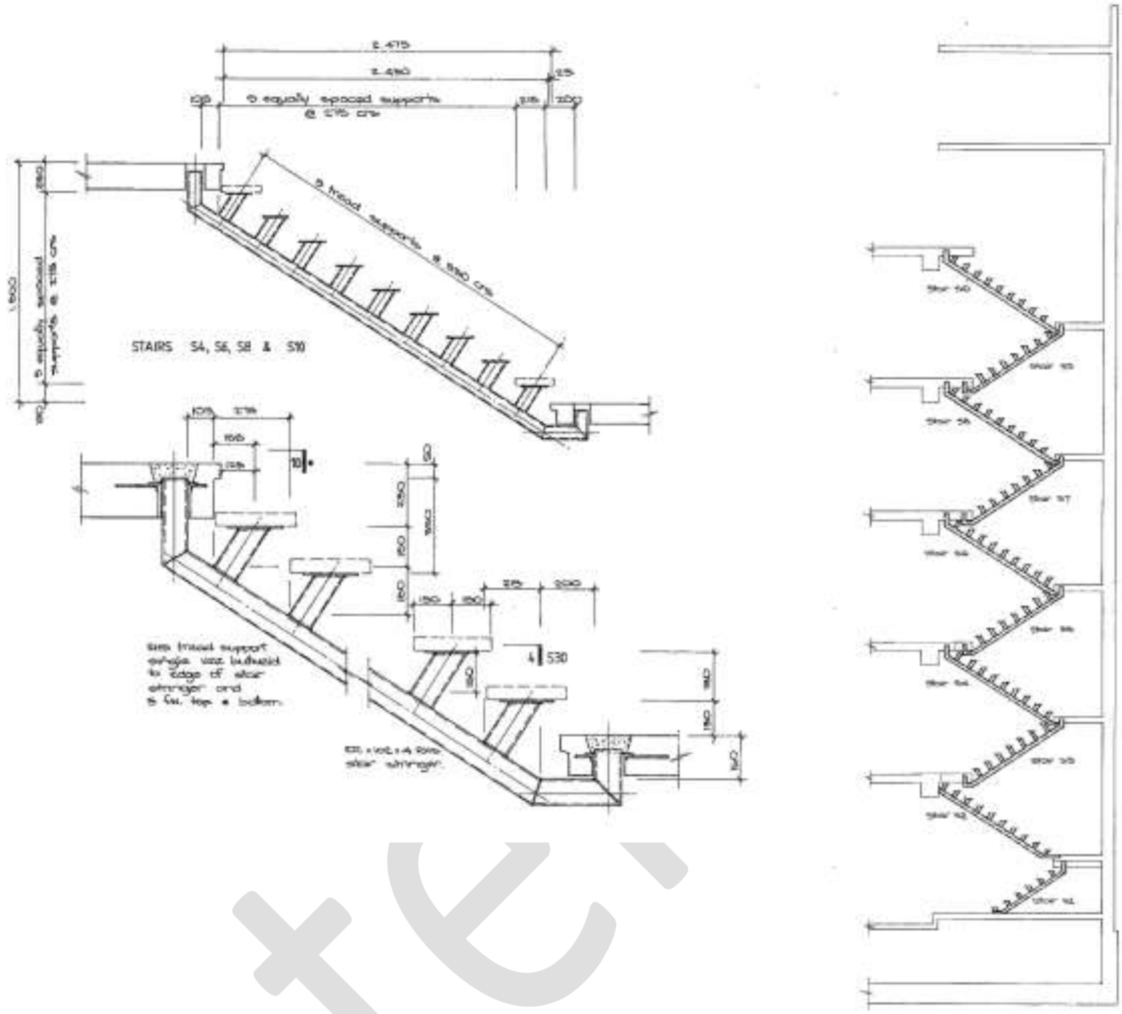

Figure 69 Line 4 to 5 Stairs and detail of Stair S8 Level 4 to 5 (extract from DENG Dwg S31)

Appendix G: Structural Specification

Portions of the DENG structural specification are included with permission to aid interpretation of the report.

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.

Figure 70 Extract from DENG Concrete Specification

b. Precast Concrete Specification

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.

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 71 Extract from DENG Pre-cast Concrete Specification