TM Consultants Ltd.
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DETAILED ENGINEERING EVALUATION

52 HAYTON ROAD

WIGRAM

CHRISTCHURCH

ISSUE B

13 May 2014

JOB NO: 130329

CONTENTS

INTRODUCTION

This report has been prepared for Body Corporate #79398 for the property at 52 Hayton Road. The report details the results found from an engineering investigation into the performance of the building in the recent earthquakes, and its likely structural capacity for future events.

The format of the investigation and the reporting is in accordance with the "Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury", Part 2. (Labelled as DEE in future references in this report)

WORK COMPLETED

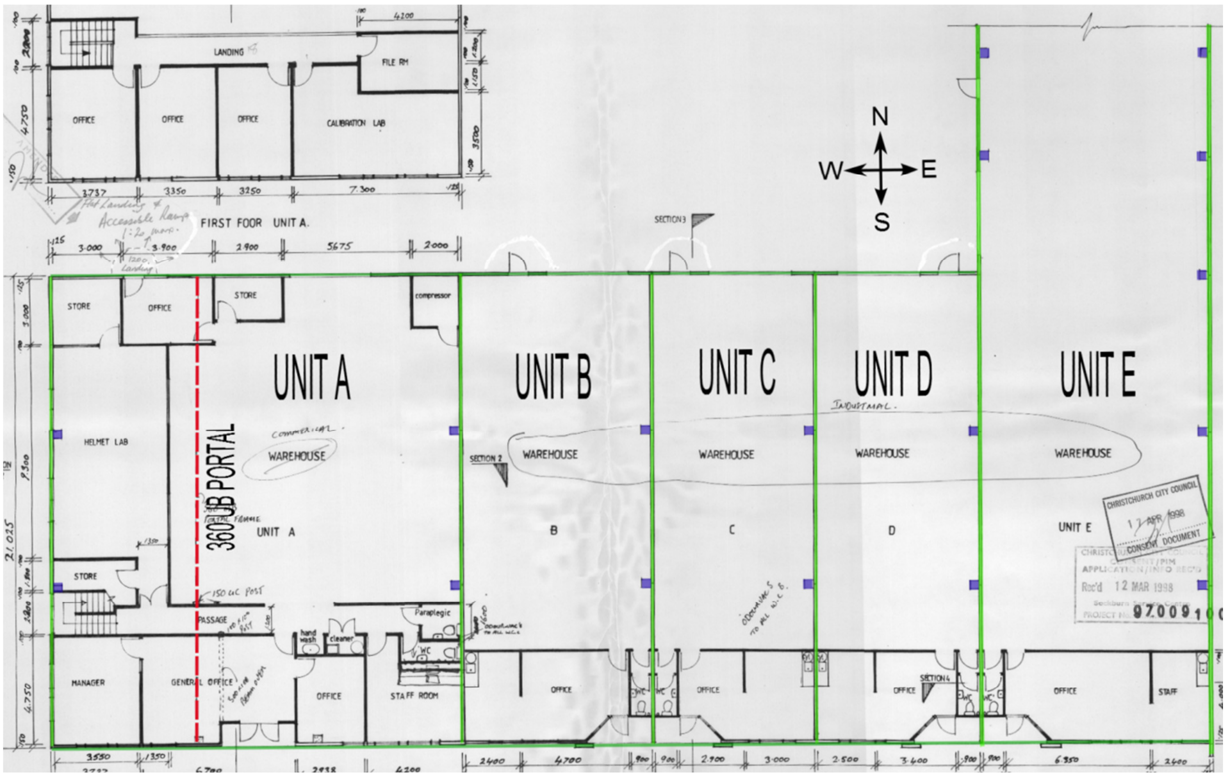
The work completed consists of:

- The gathering of existing building information. This involved looking at existing plans and checking the structure on site against these plans. The plans were found to be accurate where checked.
- A detailed damage inspection has been carried out checking damage sustained from the recent earthquakes. This included accessing the roof space to inspect for structural damage to roof bracing and purlins.
- A detailed analysis of the building has been carried out. Calculations targeting critical elements of the structure have been performed to determine the earthquake capacity of the building relative to the current code (%NBS).
- No invasive investigation was required as the structure was identifiable from the existing plans and visible on site.

BUILDING INFORMATION

Address	52 Hayton Road Wigram Christchurch 8042
Usage	Commercial- Current tenants are SAI Global, Opus Lab
No. of Storeys	2
Year of construction	1998
Building Dimensions	48m x 21m - Total floor area approx. 1139m^2 Unit A – 21m x 18m Floor Area = 378m^2 Unit B – 21m x 8.25m Floor Area = 173m^2 Unit C – 21m x 4.3m Floor Area = 90m^2 Unit D – 21m x 6.9m Floor Area = 145m^2 Unit E – 35.825m x 9.85m Floor Area = 353m^2
Foundations	200mm wide x 400mm deep unreinforced concrete perimeter strips supporting wall panels. 1000mm deep reinforced concrete post holes under concrete ground floor slab and wall panels (4/D24 vertical bars and an 6/R10 ties)
Floor	125mm deep reinforced concrete ground slab (12mm Reid Bar reinforcement at 1.0m c/c in both directions) bearing onto reinforced post holes.

Internal Structure	125mm full building height concrete tilt panels in the north/south direction divide the building into 5 separate units. Unit A contains an additional 360UB portal frame in the north/south direction. 350mm x 300mm x3.0m high reinforced concrete columns are positioned at the wall panel joint locations, approximately 7.0m from the external walls in the east/west direction.
Walls	<u>1994 Original Structure</u> 125mm thick, 6.2m – 4.2m high concrete precast internal wall panels reinforced with D10 at 150mm c/c central reinforcement in both directions. 150mm thick, 6.5m – 4.0m high concrete precast external wall panels reinforced with H12 bars around openings and D10 at 150mm c/c central reinforcement in both directions. Internal partition walls are timber framed with GIB plasterboard linings.
Roof	Zincalume Trimdek cladding on sisalation on galvanised netting on 200/18 DHS purlins at 1.2m c/c. DHS purlins span in the east/west direction between the 360UB portal frame and the concrete wall panels.
Structural System	Gravity loads are transferred from the lightweight roof structure down the portal frame and concrete wall panels into the concrete building foundations. Lateral loads are resisted in the North/South direction by the in plane actions of the concrete tilt panels and the 360UB portal frame. Lateral loads in the East/West direction are transferred through the roof purlins and resisted by the reinforced concrete wall panels and reinforced concrete columns.



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<u>GENERAL BUILDING LAYOUT – 52 Hayton Road</u>

BUILDING HOT SPOTS

The evaluation has been completed using Appendix A of the DEE guidelines.
The building has been reviewed to determine potential "hot spots" in the structural system. Being a precast concrete wall construction with a portal frame and GIB lined internal timber framed walls, the following areas are of concern:

Potential Issues	Observation of the Performance
Inadequate foundations	The foundations do not show any signs of structural damage. The floors have not settled or spread laterally indicating that the floor slab has performed well.
Brittle panel connections and/or cracked panels at the connection.	Repair work has been carried out to a number of the panel connections throughout the building.
Hard–drawn wire mesh reinforcing or inadequate reinforcing contents making panels prone to non-ductile face loading failure.	Cracking was observed in the concrete walls panels that reflect earthquake face loading, cracks spanning horizontally between the column supports. The cracks observed are of nominal thickness and indicate that the panel reinforcement is performing sufficiently and that the cracks are not of structural concern.
Inadequate shear strength of concrete wall panels.	The concrete panels have performed well resisting in plane shear actions. There has been no diagonal shear cracking around panel openings on the lateral load resisting walls.
Panel/Purlin connections	The purlin connections to the concrete tilt panels have behaved well and do not appear to have suffered any damage. All connections where checked appear in good condition.
Inadequate roof bracing	The roof members do not show any obvious signs of elongation. This indicates that the roof bracing had adequate capacity for the lateral loads experienced during the recent sequence of earthquakes.

BUILDING STRUCTURAL STRENGTH COMPLIANCE

Calculations have been carried out to determine the earthquake capacity of the building relative to the new building standards. The capacities of the following elements are as follows:

North/South Direction

- Portal frame capacity: 80%NBS
- Concrete panel face load capacity: 68%NBS
- Panel connections: 40%NBS (angle bracket capacity)
- Concrete panels in plane: 100%NBS
- Post hole capacity: 100%NBS
- Roof Purlins weak axis bending capacity: 100%NBS

East/West Direction

- Concrete panel face load capacity: 71%NBS
- Purlin compression transfer capacity: 85%NBS
- In plane strength of panels: 100%NBS
- Panel connections: 40%NBS (angle bracket capacity)
- R.C Column Capacity: 94%NBS
- R.C Column Connections: 100%NBS

SUMMARY

The building has behaved exceptionally well sustaining only non-structural cracking to a number of the concrete wall panels. The majority of the cracking has since been repaired.

The floor shows no signs of settlement or lateral spread.

Calculations concluded that the earthquake capacity of the building is approximately 40%NBS. This is governed by the angle bracket connections between the concrete wall panels. A number of the panel connections have already been replaced and repaired in previous work carried out.

A lot of the remediation work has already been carried out to return the building to its pre earthquake condition. This is stated in the damage report (Appendix 2).

The building is not classed as an earthquake prone building.

In the opinion of the writer, the building can continue to be used at full occupancy without restrictions. From the findings in this report and based on the previous performance of the panel connections it would be advise to repair or replace a number of the concrete wall panel connections. Increasing the capacity of the panel connections would increase the overall building's earthquake capacity to over 67% NBS and no further strengthening works would be required. Please refer to appendix 5 for strengthening work carried out on the 13.05.14.

Paul Sweeney

Paul Sweeney

TM CONSULTANTS LTD

Checked and approved by

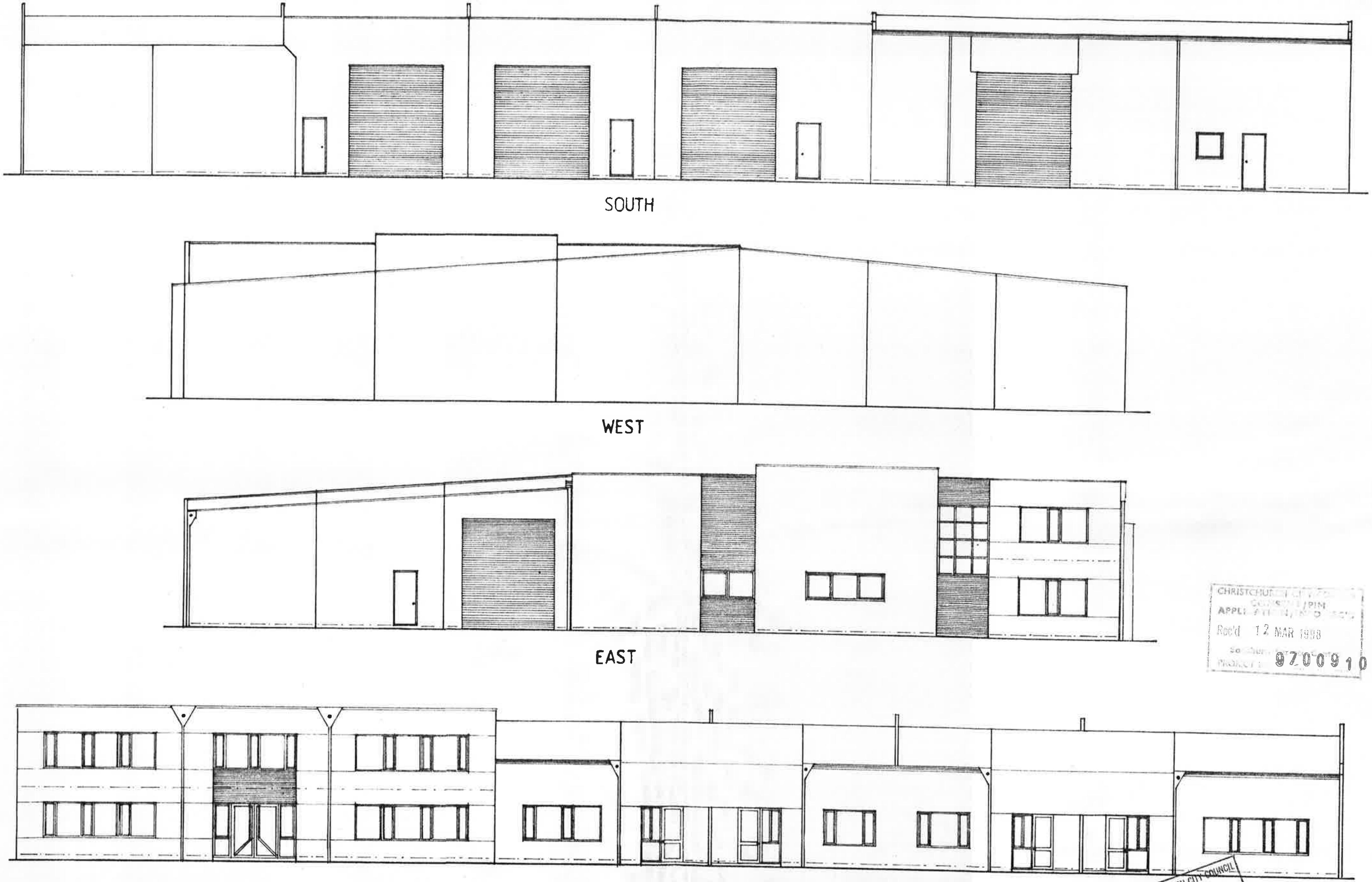
M. B. Blyth

Matthew B Blyth

CPENG # 237435

Appendix 1

Building Plans



Chris Bunn Design
301B Burwood Road, ChCh 9.
Ph. 025-327-967 Fax 03-383-1515

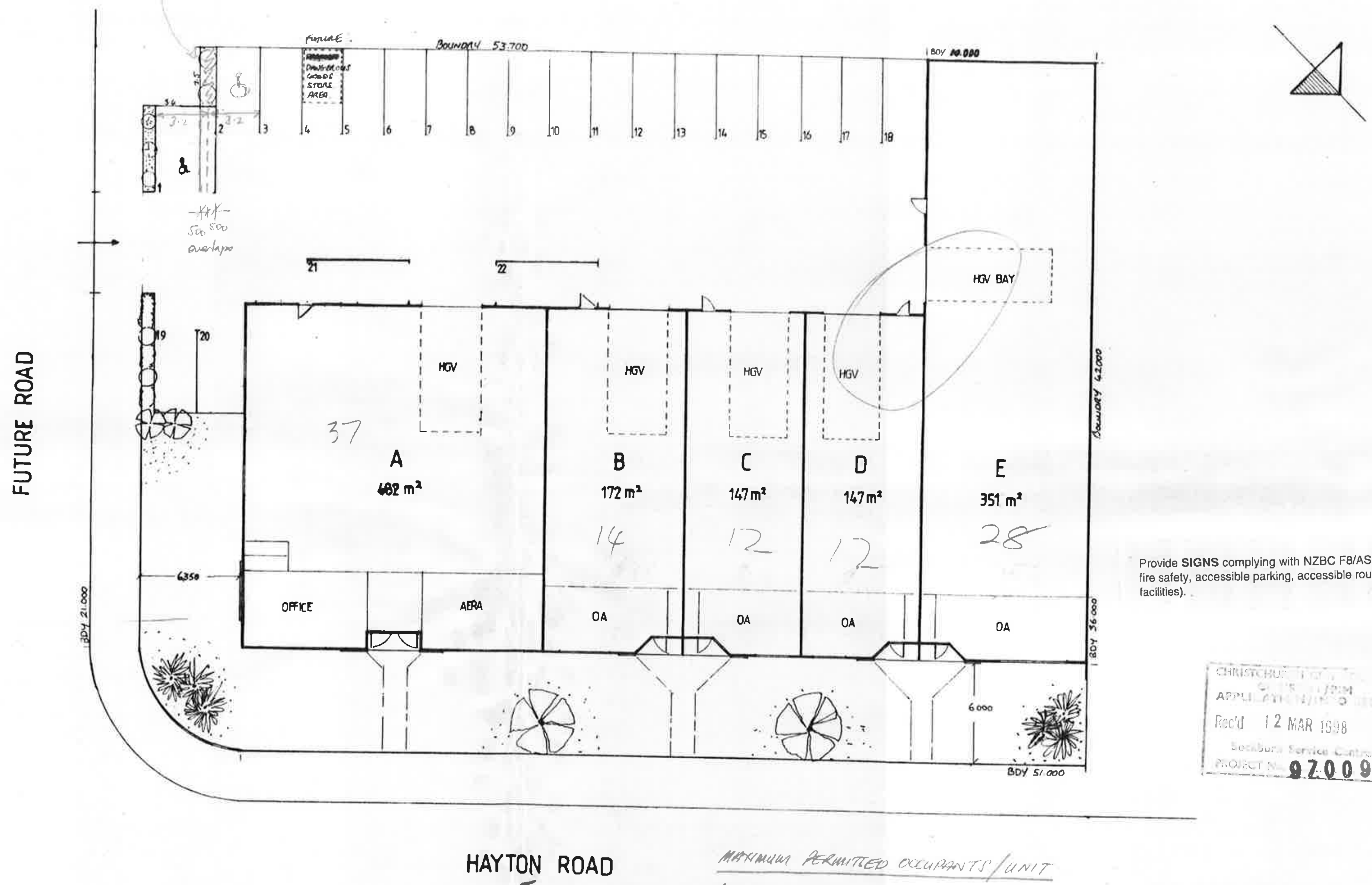
HAYTON ROAD DEVELOPMENT
LOT 33

All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.

DRAWN	CHECKED	SCALES 1:100	SHEET A1
TRACED	DATE JAN 98		SERIES OF
			REF

AMENDED
C.C.C.

delete garden; substitute sealed level surface



Provide **SIGNS** complying with NZBC F8/AS1 (e.g. for fire safety, accessible parking, accessible routes and facilities).

BUILDING AREA 1201 m²

MAXIMUM PERMITTED OCCUPANTS / UNIT

UNIT	A	=	20
	B	=	10
	C	=	10
	D	=	10
	E	=	10

AMENDED
C.C.C.

SITE AREA 2370 m²
LANDSCAPING 430 m²

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Ph. 025-327-967 Fax 03-383-1515

HAYTON ROAD DEVELOPMENT
LOT 33

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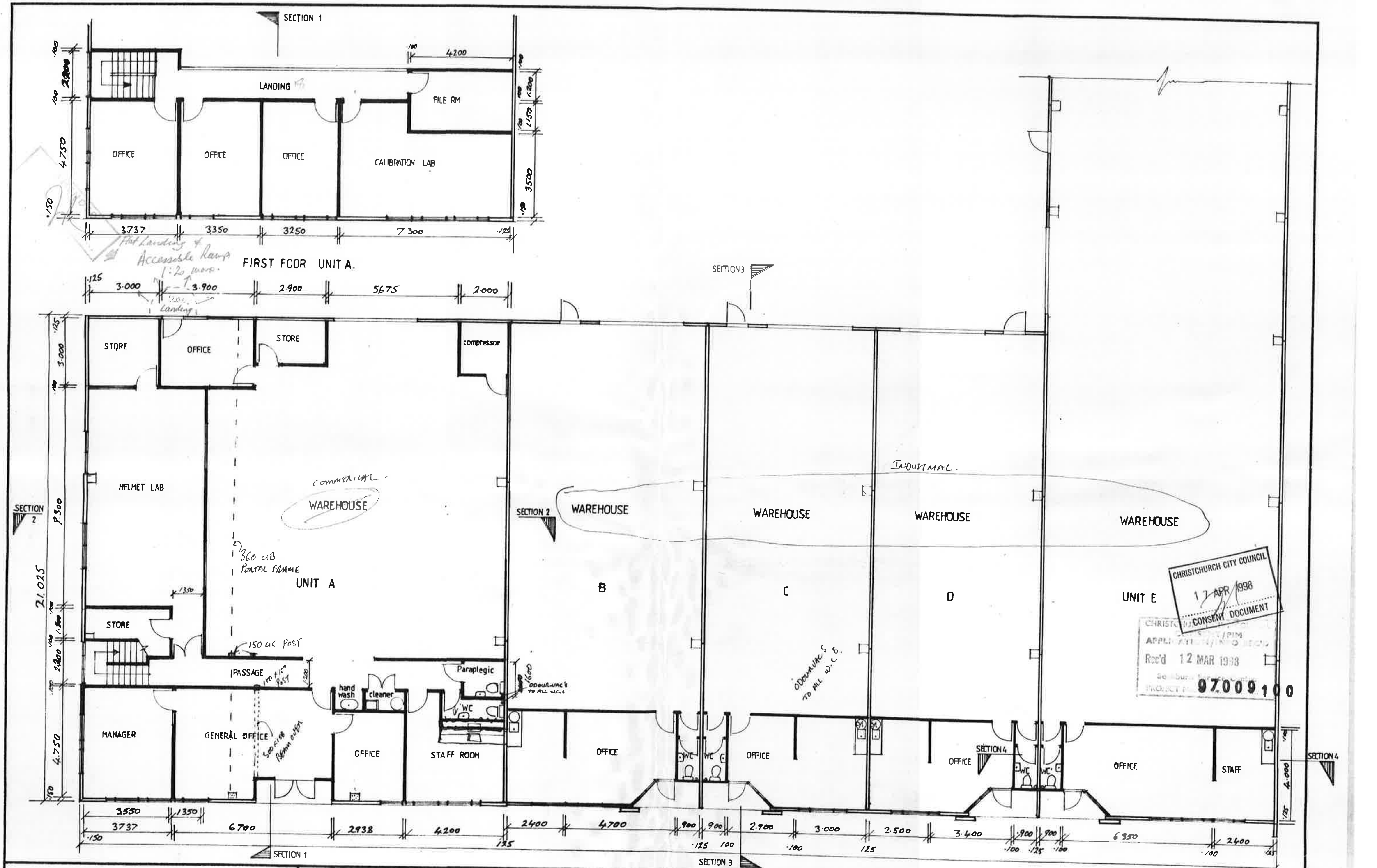
DRAWN	CHECKED	SCALES 1:200	SHEET A2
TRACED	DATE OCT 97		SERIES OF / REF 74



Chris Bunn Design

301B Burwood Road, ChCh 9
PH 025-327-967 Fax (03) 383-1515

HAYTON ROAD DEVELOPMENT
LOT 33

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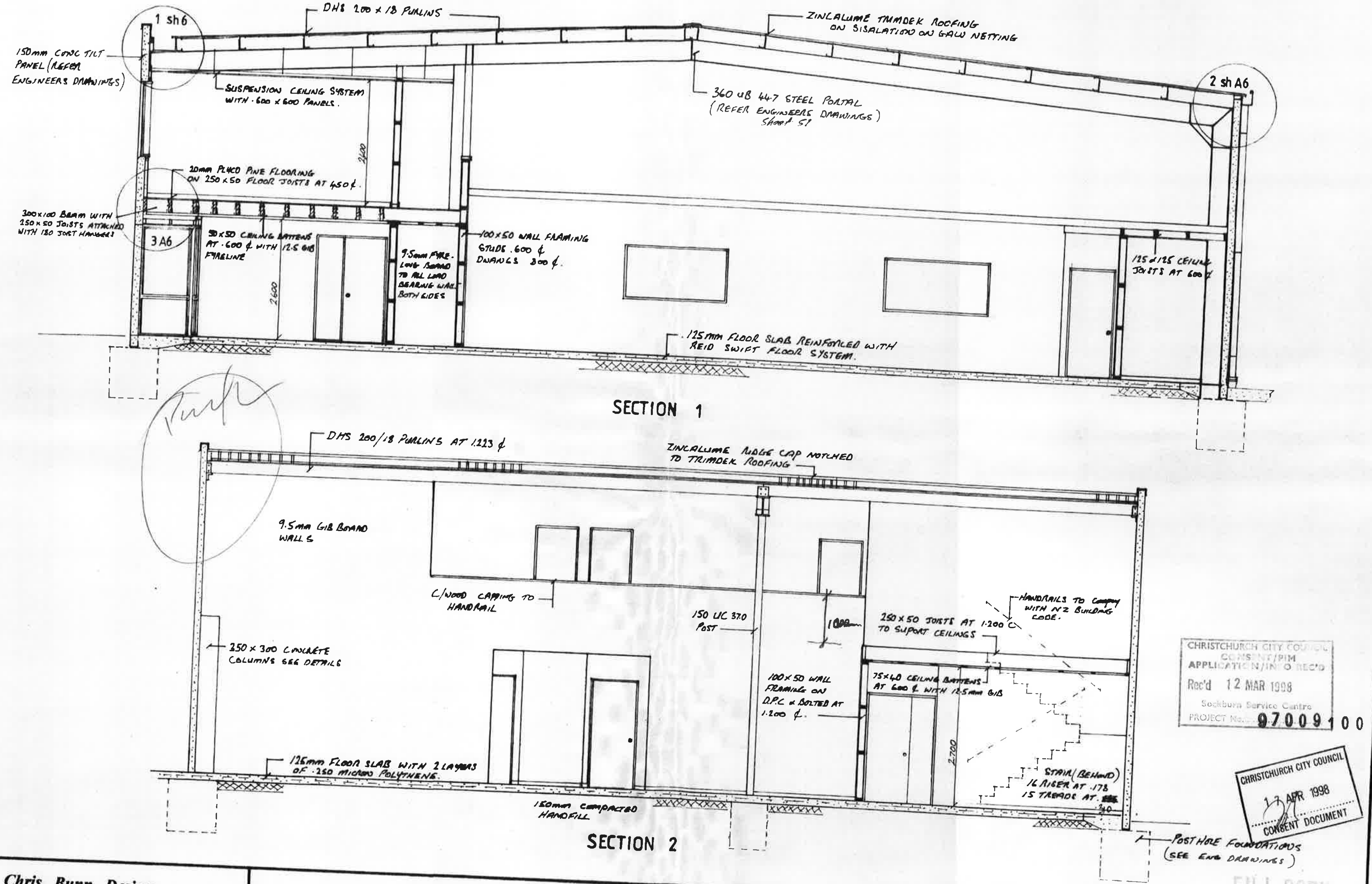
PLAN VIEW FILE COPY

DRAWN	CHECKED	SCALES 1:100	SHEET A3
TRACED	DATE JAN 98		SERIES OF / REF



Chris Bunn Design
301B Burwood Road, ChCh 9
PH 025-327-967 Fax (03) 383-1515

HAYTON ROAD DEVELOPMENT
LOT 33

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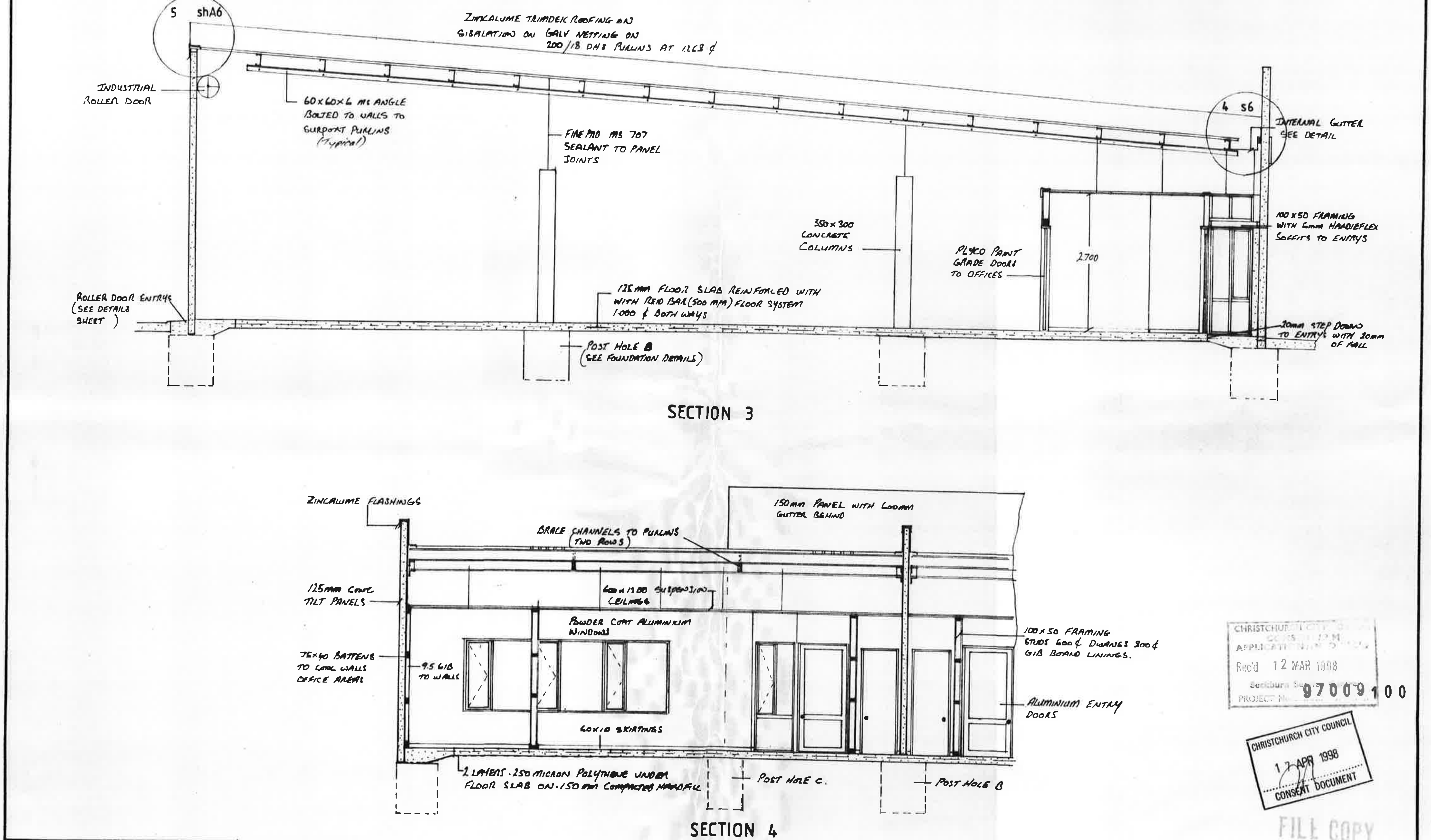
SECTIONS 1,2	SHEET A4
DRAWN	CHECKED
TRACED	SCALES



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PH 025-327-967 Fax (03) 383-1515

HAYTON ROAD DEVELOPMENT
LOT 33

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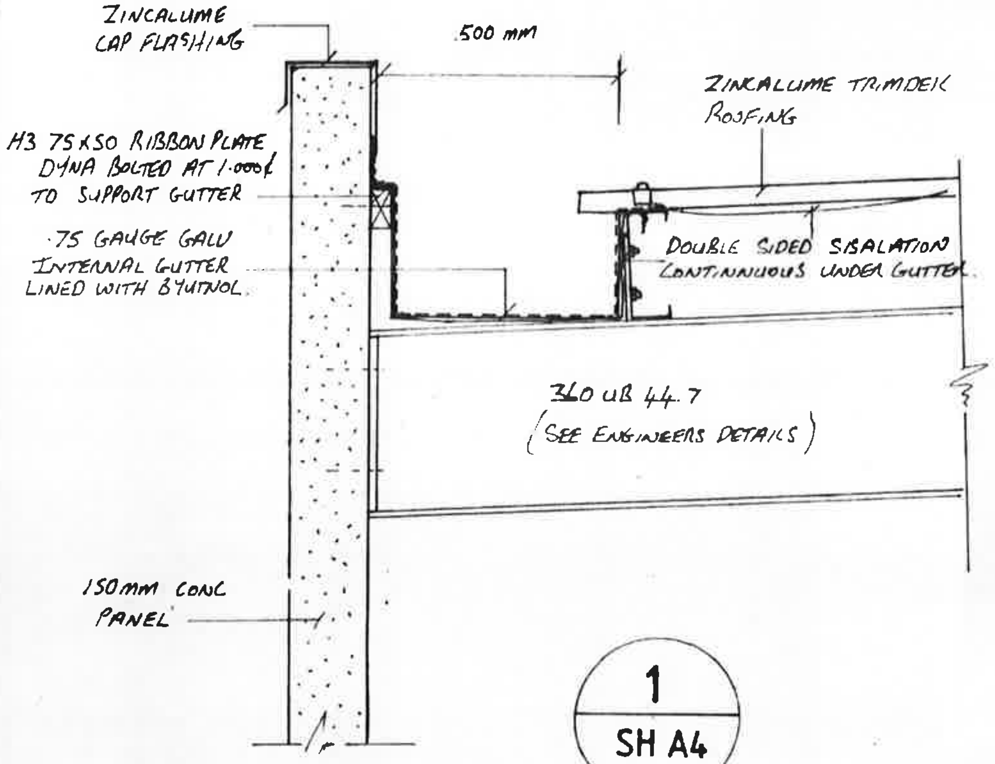
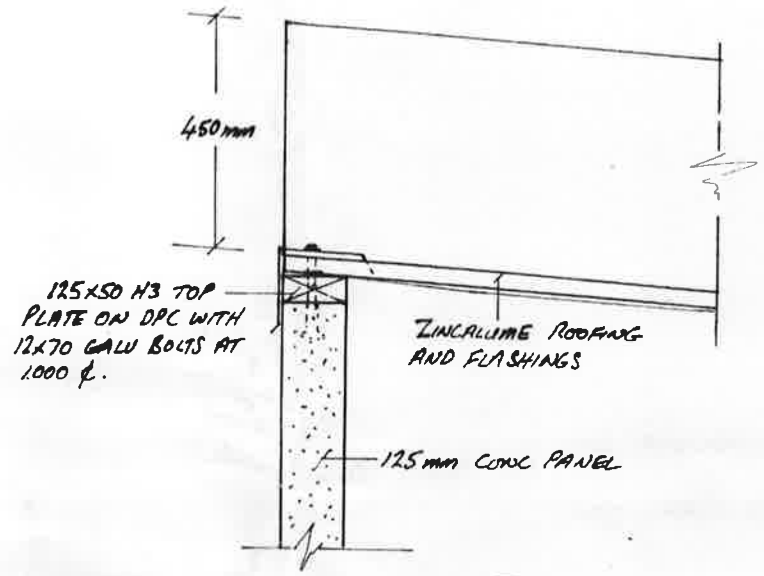
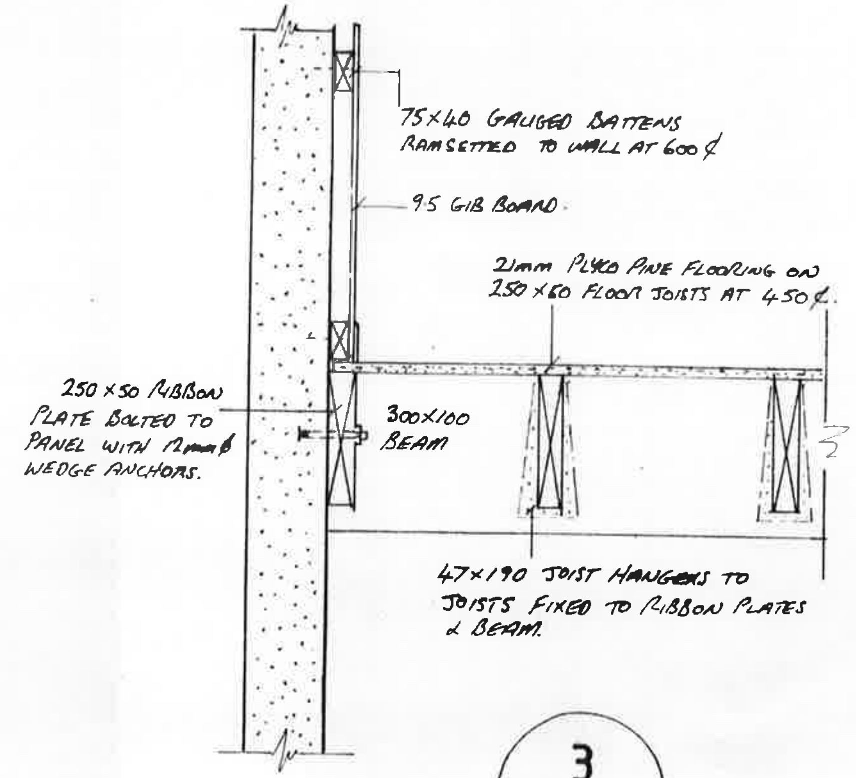
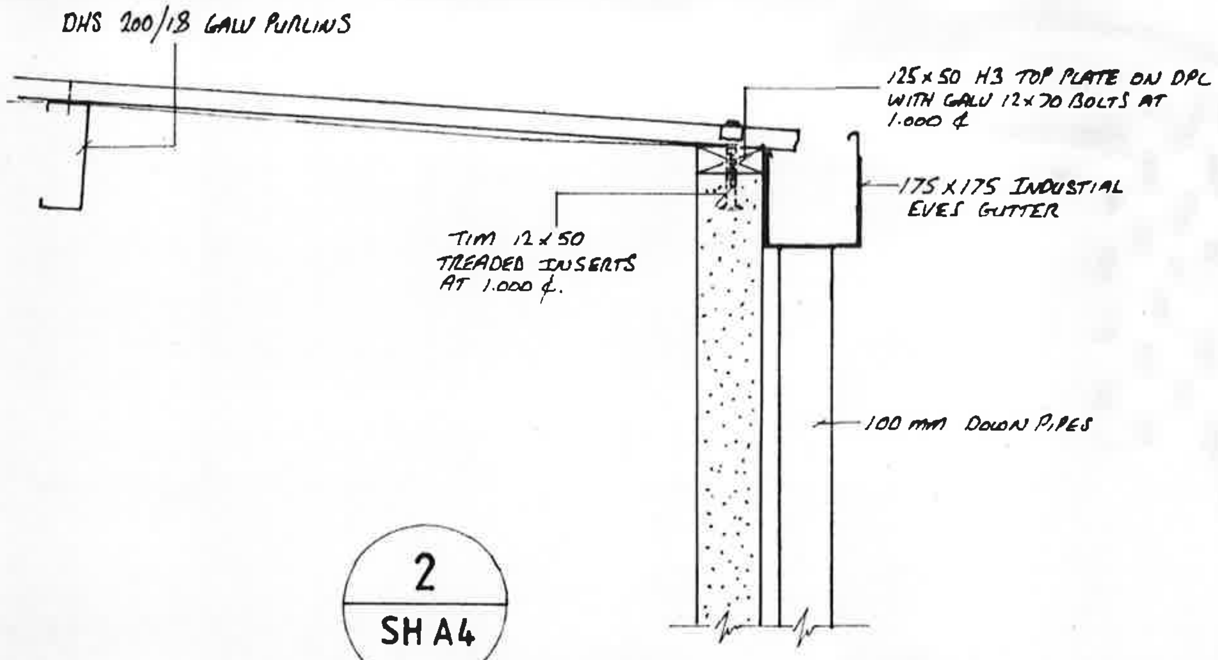
SECTIONS 3,4

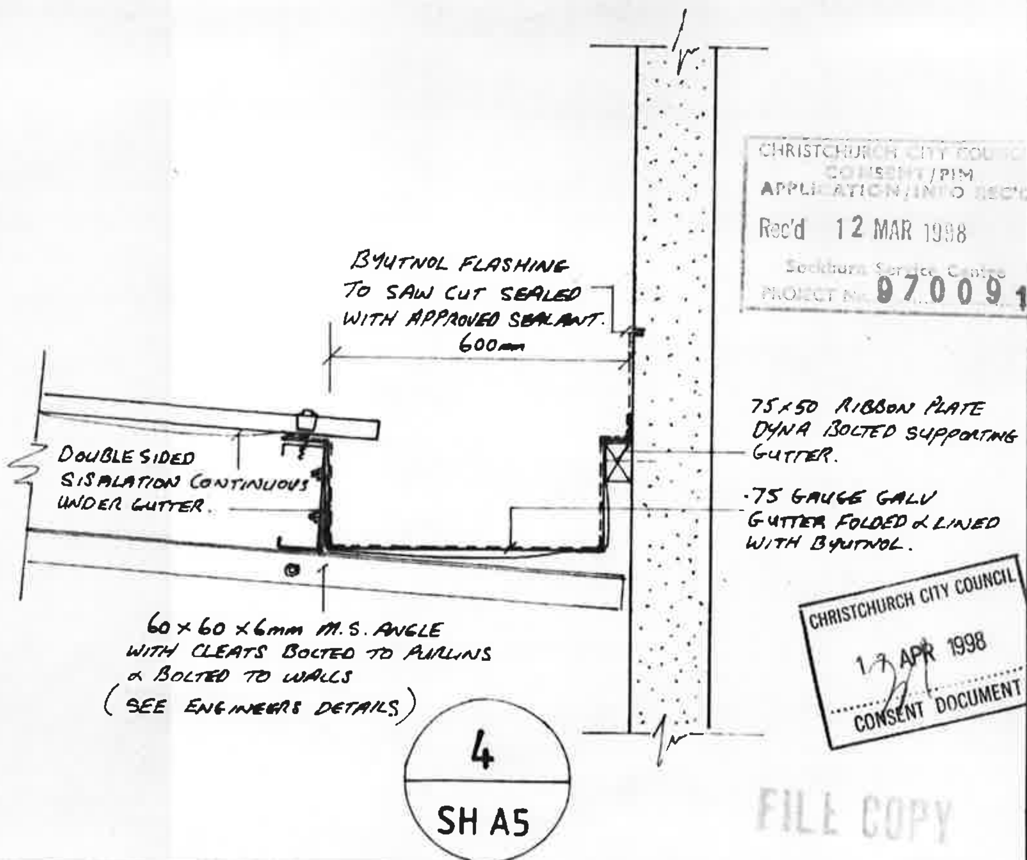
SHEET	A5
DRAWN	
TRACED	
CHECKED	
DATE	FEB 98
SCALES	1:50
SERIES OF	
REF	74











Chris Bunn Design
301B Burwood Road, ChCh 9
PH 025-327-967 Fax (03) 383-1515

HAYTON ROAD DEVELOPMENT
LOT 33

All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.

SECTION DETAILS.

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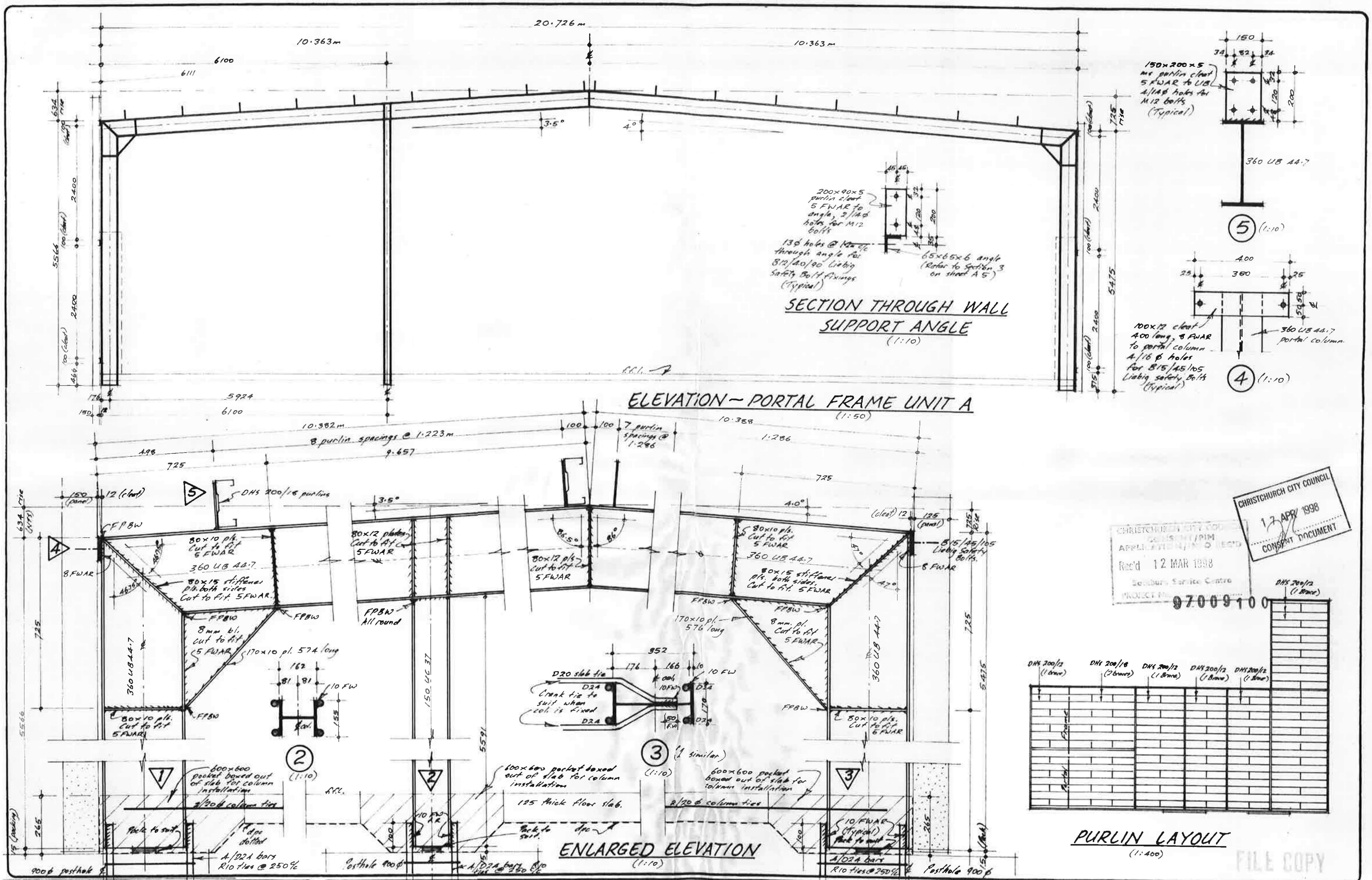
TRACED	DATE FEB 98		SERIES OF / REF 74



ENGINEERING & BUILDING ADVISORY SERVICE

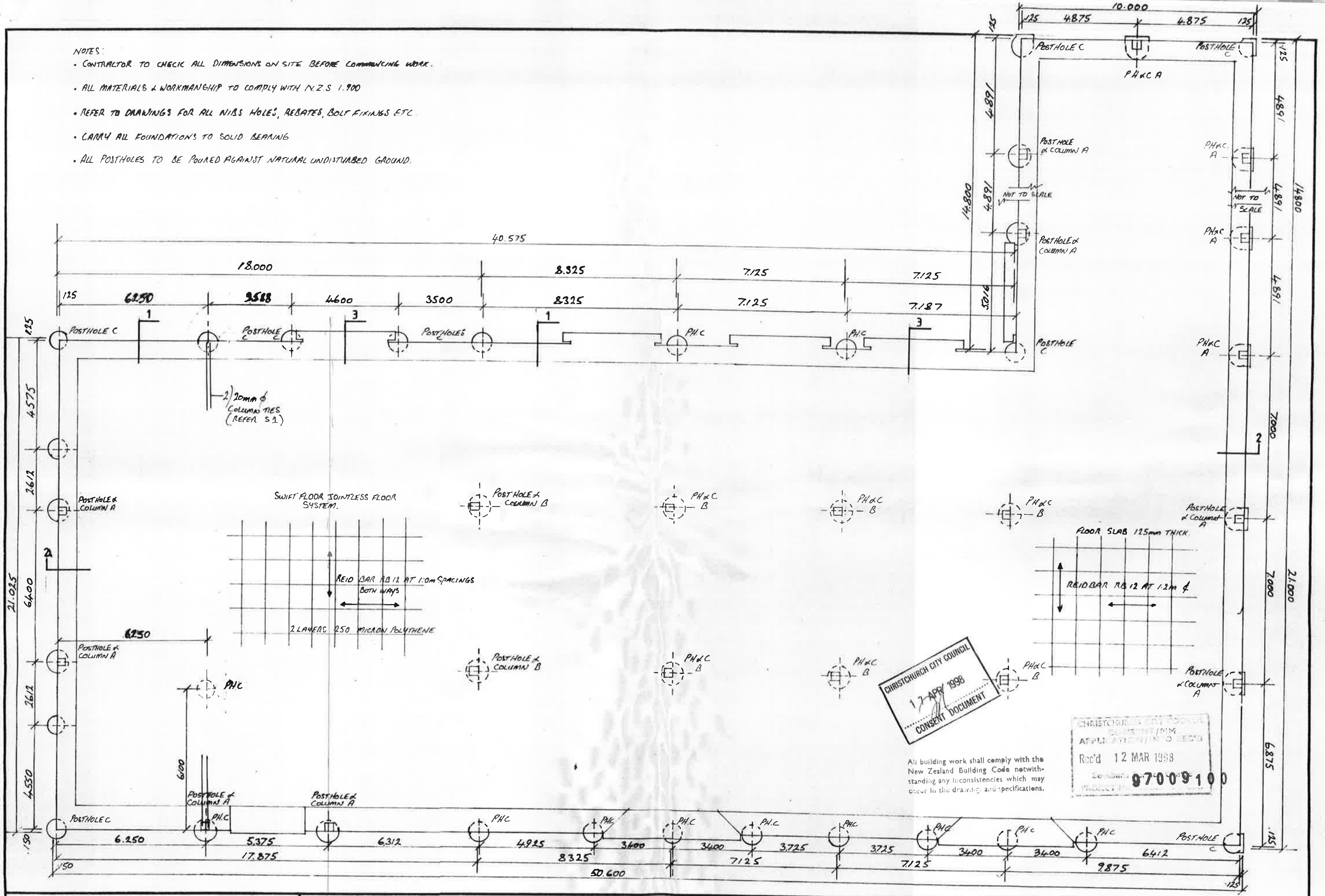
246 HIGHSTED ROAD, CHCH. CELLULAR 025-322-914

HAYTON ROAD DEVELOPMENT ~ LOT 33

All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.

Portal Frame Details Unit A			SHEET S1
DRAWN P. Nichols	CHECKED	SCALES As shown	SERIES OF
TRACED	DATE March 1998		REF

NOTES:
- CONTRACTOR TO CHECK ALL DIMENSIONS ON SITE BEFORE COMMENCING WORK.
- ALL MATERIALS & WORKMANSHIP TO COMPLY WITH N.Z.S 1.900
- REFER TO DRAWINGS FOR ALL NIBS HOLES, REBATES, BOLT FIXINGS ETC
- CARRY ALL FOUNDATIONS TO SOLID BEARING
- ALL POSTHOLES TO BE POURED AGAINST NATURAL UNDISTURBED GROUND.



CHRISTCHURCH CITY COUNCIL
17 APR 1998
CONSENT DOCUMENT

All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.

CHRISTCHURCH CITY COUNCIL
CONSENT/PIM
APPLICATION/INFO RECD
Rec'd 12 MAR 1998
97009100

ENGINEERING & BUILDING ADVISORY SERVICE

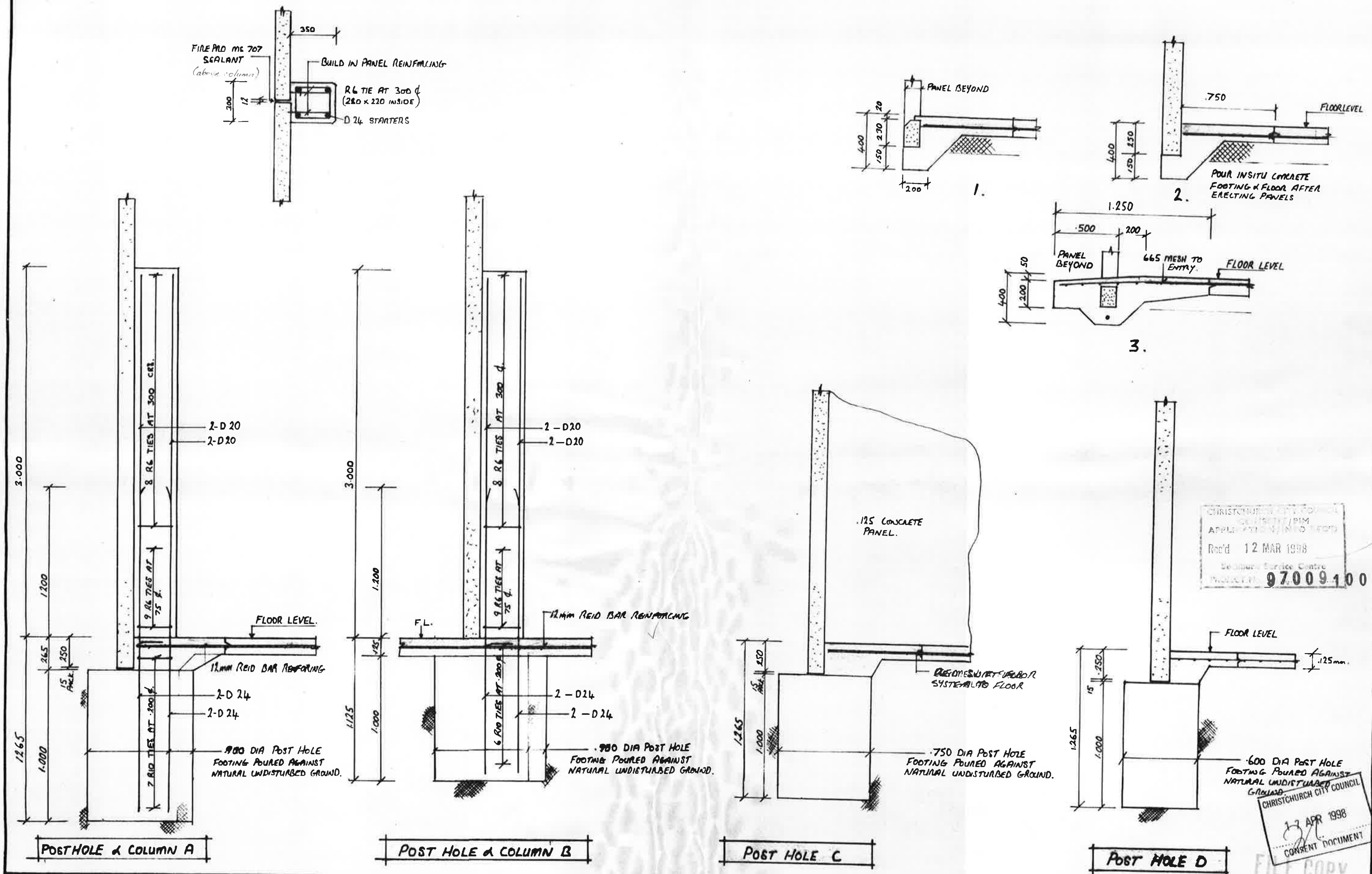
246 HIGHSTED ROAD, CHCH. CELLULAR 025-322-914

HAYTON ROAD DEVELOPMENT
LOT 33

FILE COPY

FOUNDATION PLAN

SHEET	DRAWN	CHECKED	SCALES	SERIES OF
2	TRACED	DATE JAN 98	1:100	REF 72



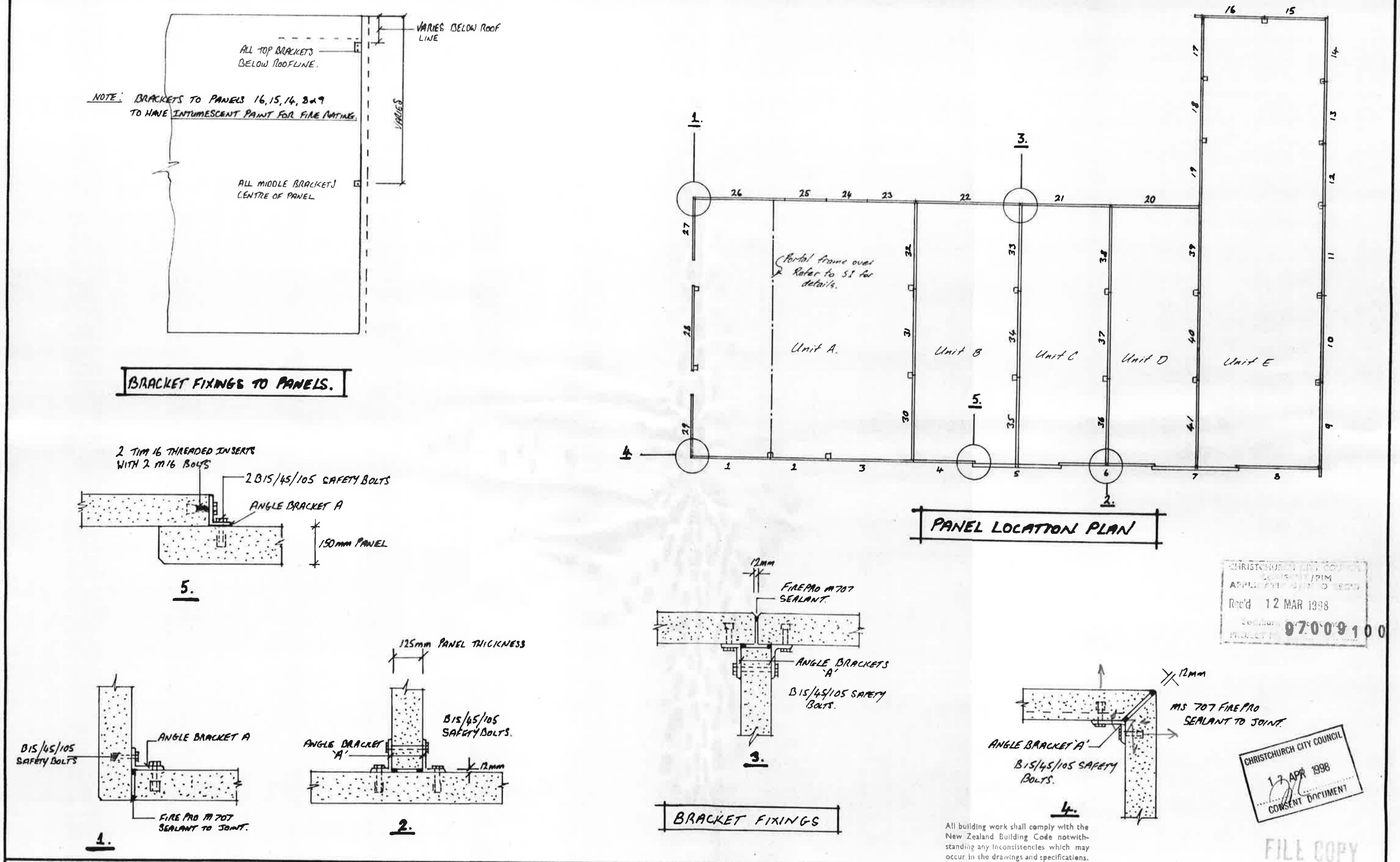
ENGINEERING & BUILDING ADVISORY SERVICE
246 HIGHSTED ROAD, CHCH. CELLULAR 025-322-614

HAYTON ROAD DEVELOPMENT
LOT 33

All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.

FOUNDATION & COLUMN DETAILS.

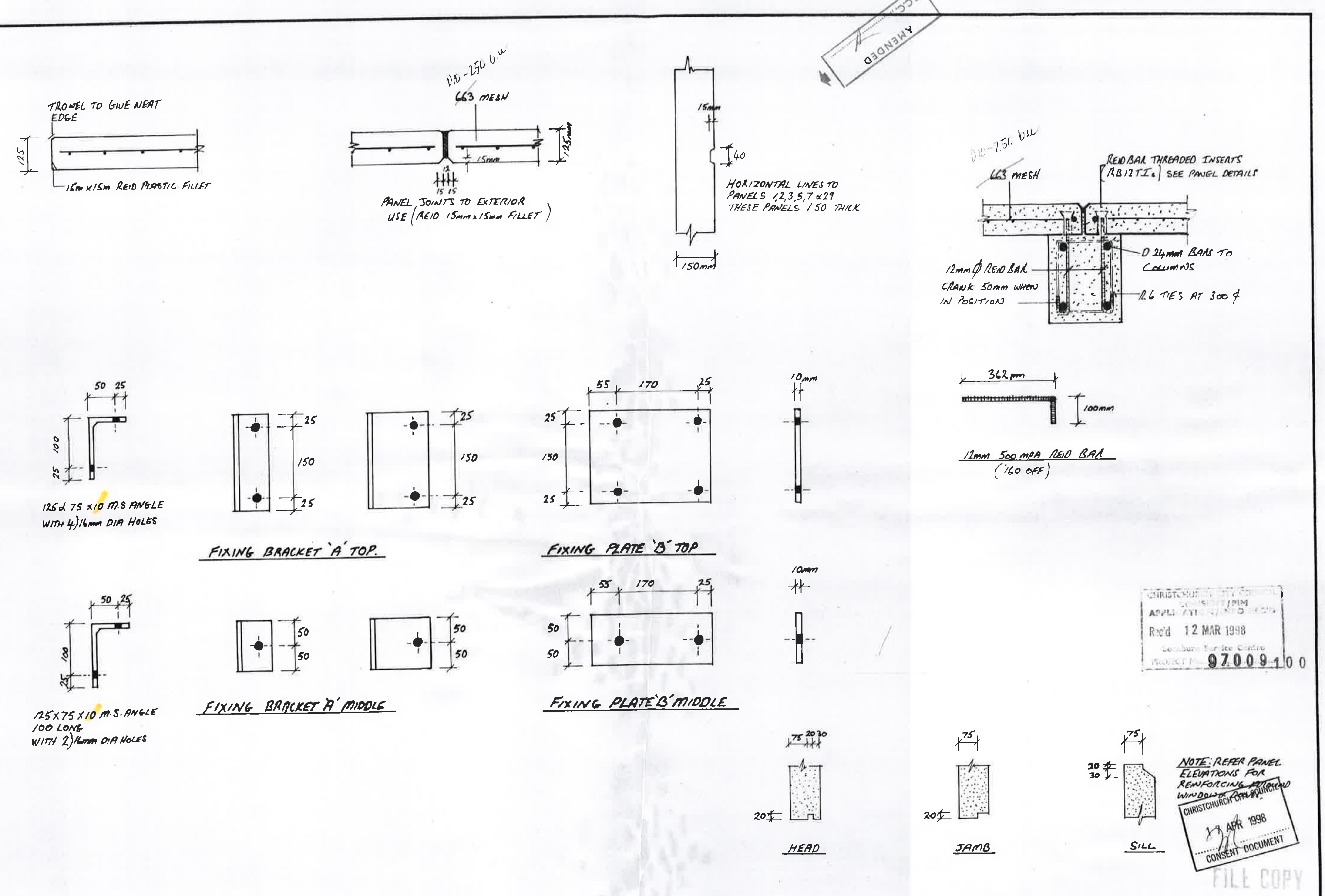
SHEET	3
DRAWN	
CHECKED	
DATE	FEB 98
SCALES	1:20
TRACED	
SERIES OF	
REF	74



All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.

HAYTON ROAD DEVELOPMENT
LOT 33

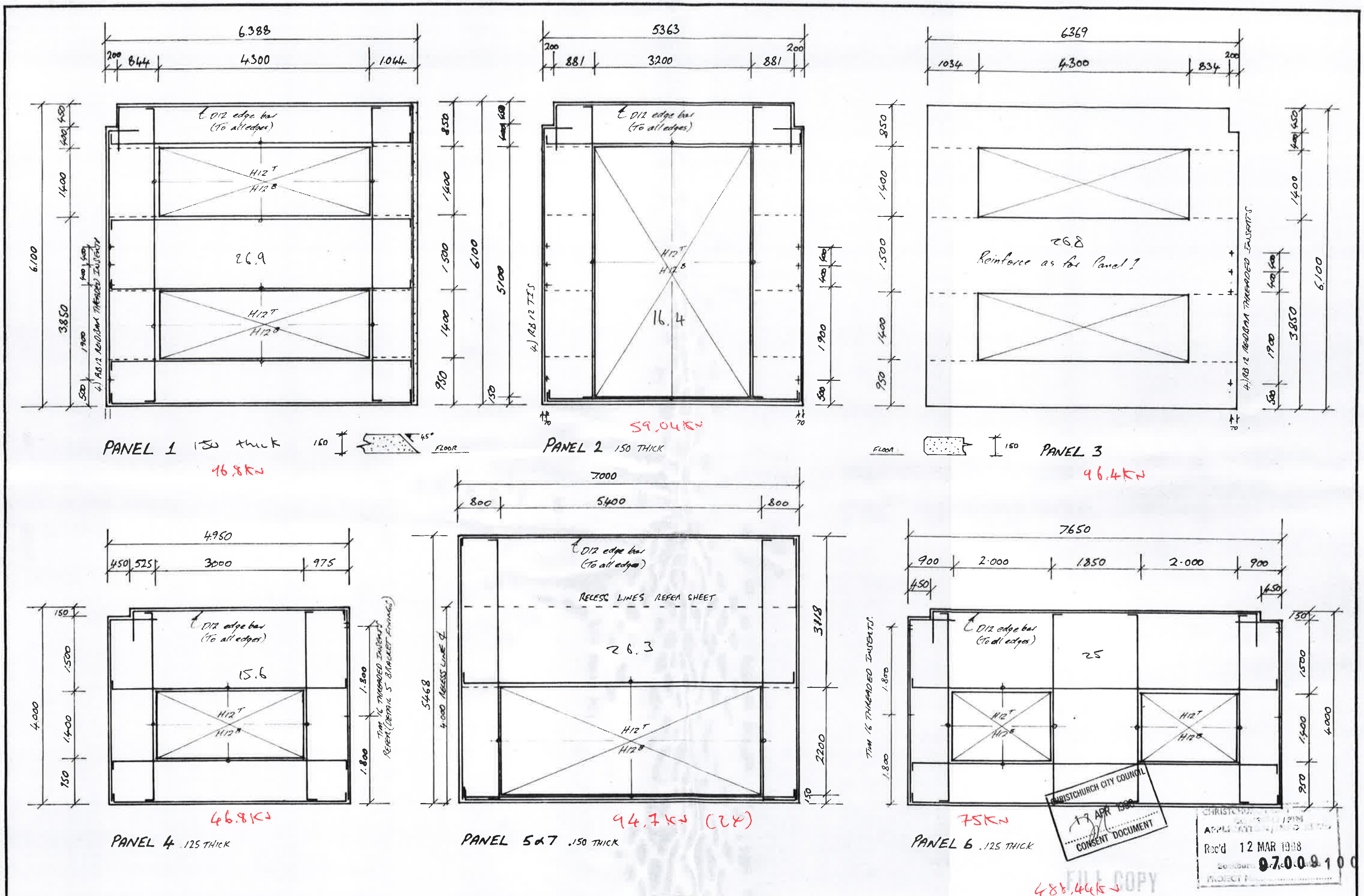
PANEL LOCATION PLAN & DETAILS & BRACKET FIXINGS.			SHEET 4
DRAWN	CHECKED	SCALES 1:200 1:10 1:50	SERIES OF
TRACED	DATE JAN 98		REF 74



HAYTON ROAD DEVELOPMENT
LOT 33

All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.

BRACKETS PLATES RODS & WALL PANEL DETAILS.			SHEET 5
DRAWN	CHECKED	SCALES 1:10 1:5	SERIES OF
TRACED	DATE FEB 98		REF 74

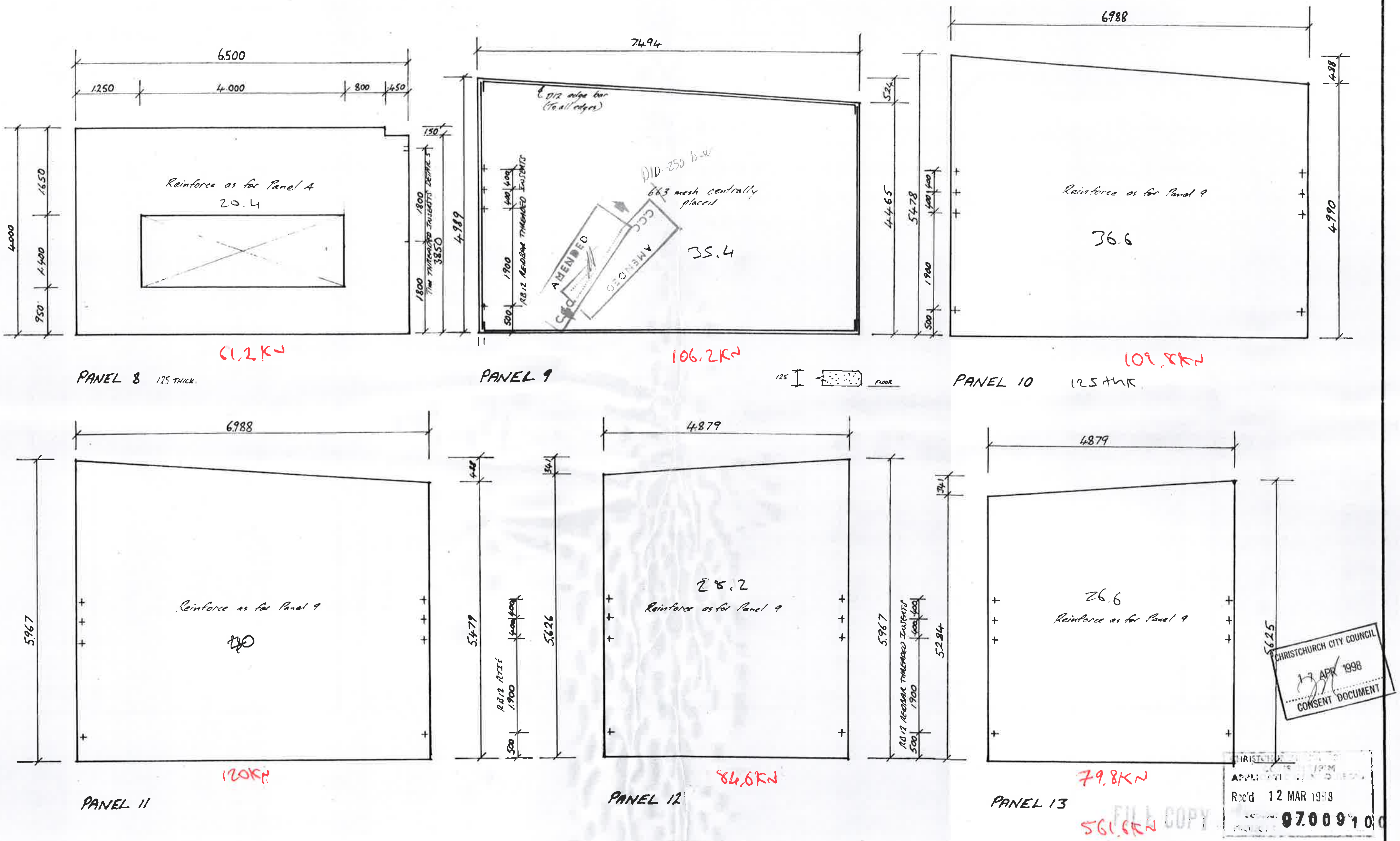


ENGINEERING & BUILDING ADVISORY SERVICE
247 HIGHSTED ROAD, CHCH. CELLULAR 025-322-914

HAYTON ROAD DEVELOPMENT
LOT 33

All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.

PANELS 1, 2, 3, 4, 5, 6, 7.			SHEET 6
DRAWN	CHECKED	SCALES 1:50	SERIES OF
TRACED	DATE JAN 98		REF 74



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PANELS 8,9,10,11,12,13			SHEET 7
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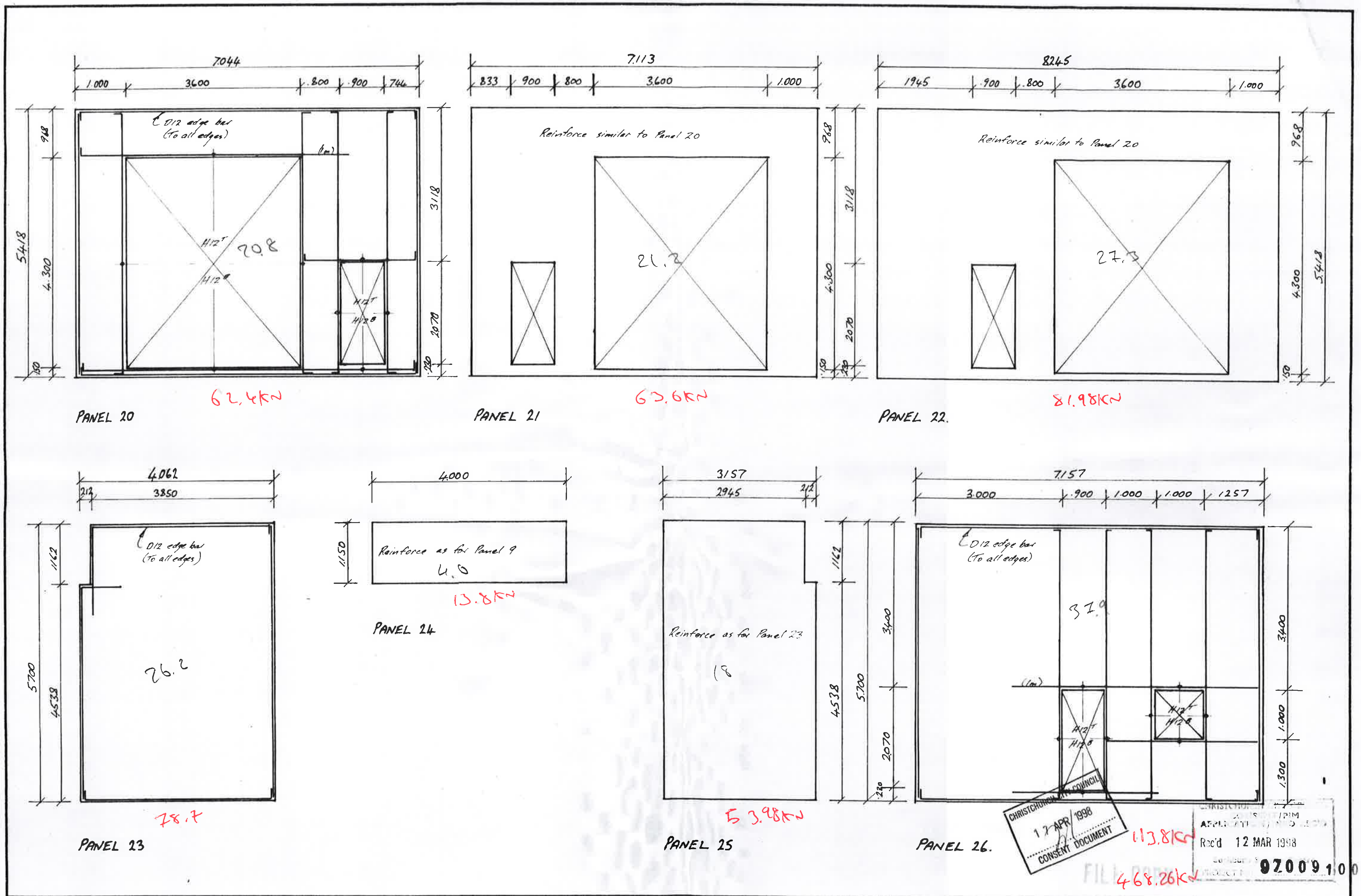
TRACED	DATE JAN 98		REF 74



All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.

PANELS 20, 21, 22, 23, 24, 25, 26.			SHEET 9
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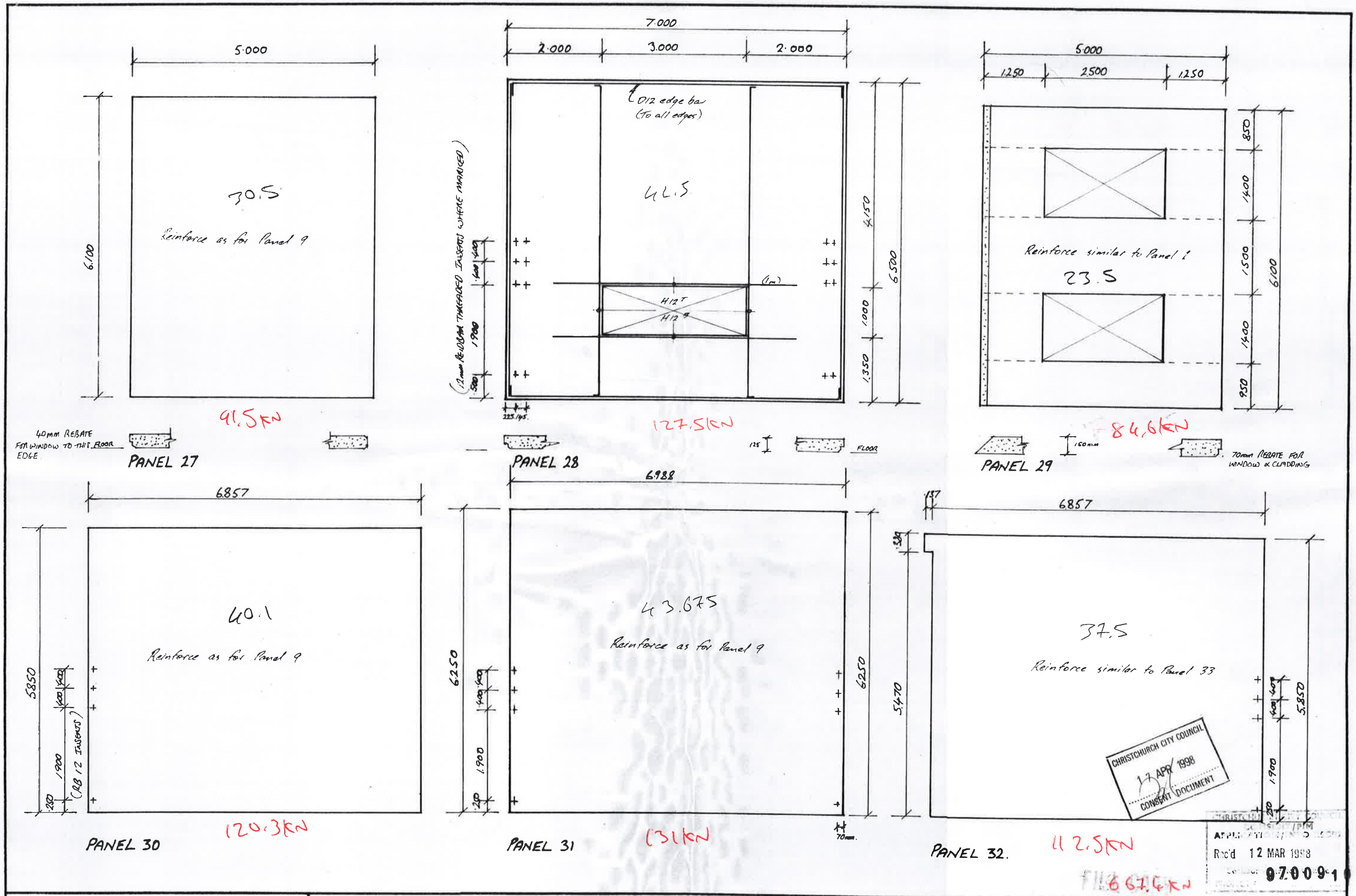
TRACED	DATE JAN 98.		REF 74



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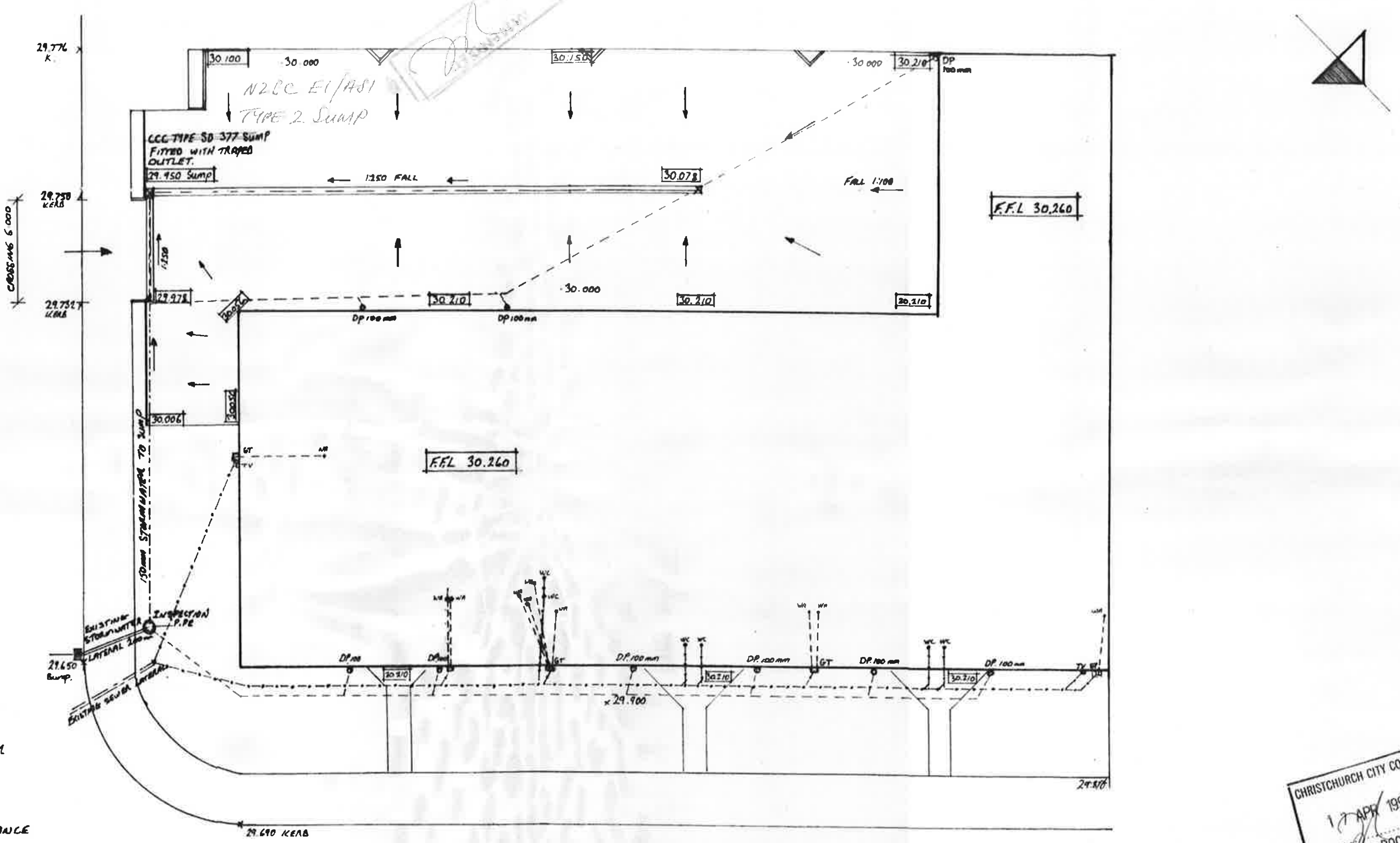
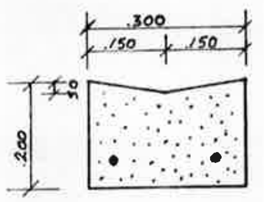
All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.

PANELS 27, 28, 29, 30, 31, 32.

DRAWN	CHECKED	SCALES 1:50	SHEET 10
TRACED	DATE JAN 98		SERIES OF / REF 74





CHANNEL 1:10

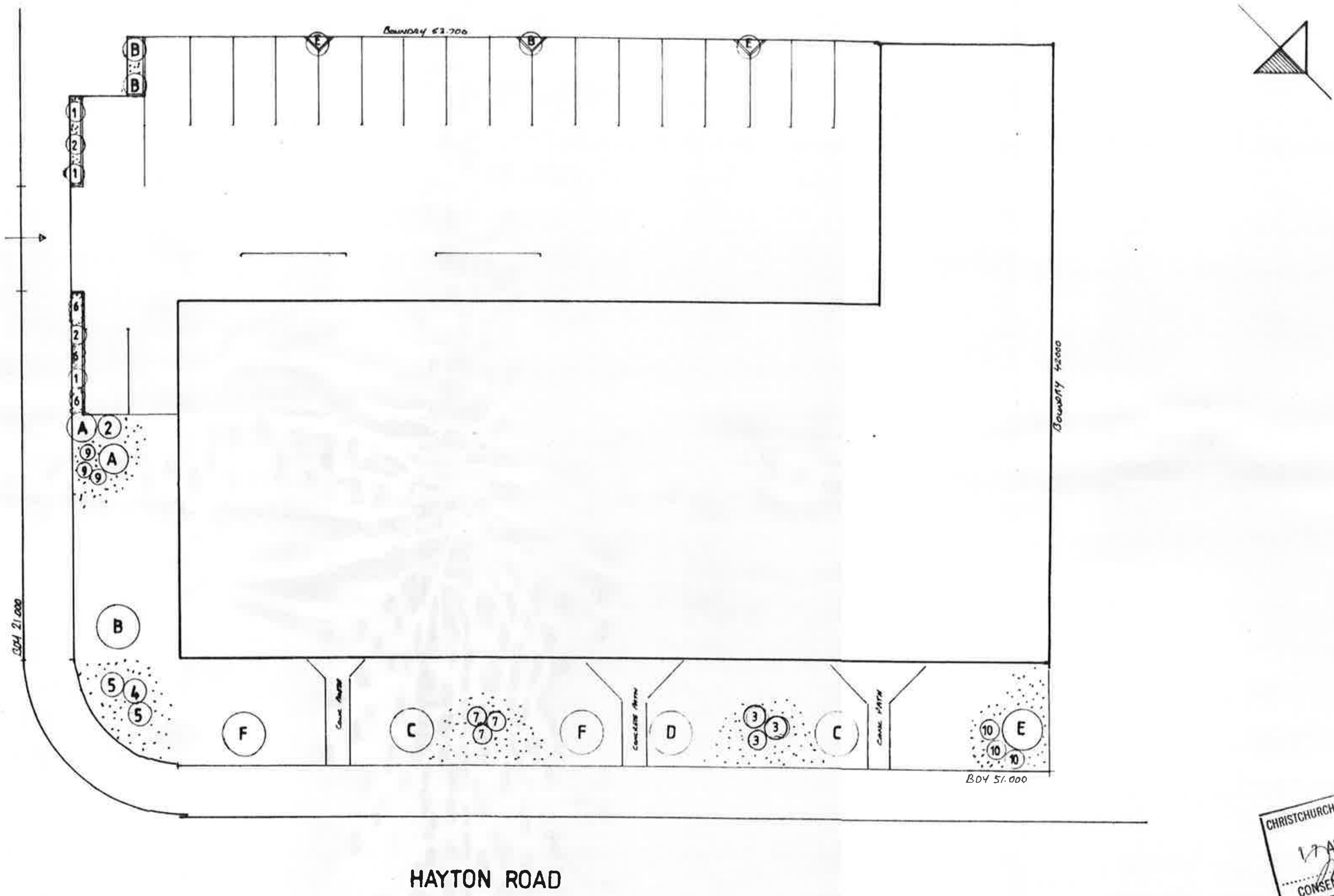
× 29.900 DENOTES AN EXISTING LEVEL
30.210 DENOTES FINISHED SURFACE LEVEL
· SUMPS TO HAVE HEAVY DUTY GALV LIDS.
· YARD CONSTRUCTION .125mm 25mpa CONC.
.150mm COMPACTED HARDFILL

—·—·— SEWER 110mm ⌀
—·—·— 150 uPVC GRADE R/40 STORMWATER
- - - - - - 110mm ⌀ STORMWATER
○ DOWNPIPES 100mm ⌀

ALL STORMWATER & SEWERAGE TO BE IN ACCORDANCE WITH THE N.Z. BUILDING CODE.

HAYTON ROAD

LOT 33 DP 76565 AREA 2376	HAYTON ROAD DEVELOPMENT LOT 33	All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.	DRAINAGE PLAN	SHEET 12
			DRAWN / CHECKED / SCALES 1:200 / TRACED / DATE FEB 98	REF 74



TREES

A. Plagianthus Betulinus
B. Pittosporum Tenufolium
C. Betula Pendula
D. Sophora Tetraptera
E. Pittosporum Eugenoides
F. Dodonaesa Viscosa

SHRUBS AND TREES

1. Pittosporum Tenufolium
2. Pittosporum Mountain Green
3. Choiysa Ternata
4. Phormium Tenax Rubrum
5. Phormium Surfer Boy
6. Coprosma Kirki
7. Coleonema Sunset Gold
9. Carex Testacea
10. Carex Comans Bronze
░ Soil and Bark Mulch

SITE AREA 2376 m²
LANDSCAPING 424 m²

HAYTON ROAD DEVELOPMENT **LOT 33**	All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.	LANDSCAPE PLAN.	SHEET 13
DRAWN	CHECKED	SCALES 1:200	SERIES OF
TRACED	DATE FEB 98		REF 74

Appendix 2

Damage Report & Levels Survey

SITE REPORT

Attn: Body Corporate #79398 attn. Chris Bunn

SR No : 01
File No : 130329
Date : 26/03/2014
Project : 52 Hayton Road

D.E.E. POST EARTHQUAKE INSPECTION

The building was designed in 1998 by Chris Bunn Design and engineered by Engineering & Building Advisory Service. The building is subdivided into 5 units and is constructed of concrete precast tilt panel with a portal frame spanning north to south in unit A. The lateral load resisting structure in the north to south direction is resisted by the in plane actions of the concrete tilt panels and the 360UB portal frame.
Lateral loads in the east to west direction are transferred through the roof purlins and resisted by the reinforced concrete wall panels and reinforced concrete columns.
The wall panels sit on concrete edge beam foundations and on post hole foundations. The floor is a 100mm thick concrete slab. Internal partition walls are timber framed with GIB board linings.
The roof structure comprises of Zincalume trimdek roofing on sisalation on galvanised netting on 200/18 DHS purlins at 1.2m c/c.
DHS purlins span in the east/west direction between the 360UB portal frame and the concrete wall panels.
The inspection was focused on structural elements. The building has been inspected inside and out. The Issues arisen from the inspection are:

LOCATION	DAMAGE	RECOMMENDED REPAIR
Foundations:	No damage was observed to the edge foundation beam where inspection was possible. There is no evidence to suggest damage has occurred to the panel foundation beam or post holes. Given the overall condition of the building after the earthquakes, the foundations are assumed to be intact.	No Repair Recommended
Floor:	The floors should no sign of settlement or lateral spread. No damage was observed to interior floor surfaces.	No Repair Recommended
Roof:	The roof structure was accessed and the purlin connections inspected where the roof load is delivered into the wall panels. The roof purlins seem to be in sound structural condition and the purlin connections appear undamaged.	No Repair Recommended
Walls/Linings:	The concrete panels have sustained minor cracking during the 2010 and 2011 earthquakes. All cracks are less than 5mm in width and are not of structural concern.	Cracking to precast panels requires grinding out and epoxy injection to repair. The surface of the panels is then to be refinished to return to pre-earthquake conditions.
	Damage has occurred to sealant joints between concrete panels due to the shaking movement.	Remove sealant and replace to repair.
	Minor cracking and superficial damage to internal wall Gib walls and linings.	'V' out the crack, re-plaster and paint to repair.

Panel Connections	A panel connection at rear of unit D and through to unit E has pulled away from concrete wall panels. Damage has occurred to the connections between the portal frame and concrete wall panels and to a panel to panel connection for the spandrel over the roller door in unit A.	Connection has been repaired Repair work has been carried out to these connections, new steel plates have been fitted as of the 19 August 2011. These connections have performed well in the subsequent earthquakes.
Exterior Surroundings:	No damage noted	No Repair Recommended

Summary

The building has performed very well sustaining only minor cracking to the wall panels in the recent sequence of earthquakes. Repair work has been carried out to a number of wall panel connections damaged in previous earthquakes. The floor shows no sign of settlement or lateral spread. There is minor cosmetic damage to some internal wall linings, most of which has been repaired. Relatively easy minor cosmetic repairs are required to restore the building back to its pre-earthquake condition.

Signature: Paul Sweeney

Appendix 3

DEE Analysis

FILE NAME: 52 HAYTON ROAD

DATE: 20.02.14 FILE No: 130329

DESIGNER: PS SHEET No: 01

TM CONSULTANTS
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LOADING

Roof: $G = 0.4\ kN/m^2$
$Q = 0.25\ kN/m^2$

internal G.b walls: $G = 0.4\ kN/m^2$

125 conc panels: $G = 3.0\ kN/m^2$

150 conc panels: $G = 3.6\ kN/m^2$

Warehouse 100 floor slab: $G = 2.4\ kN/m^2$
$Q = 5.0\ kN/m^2$

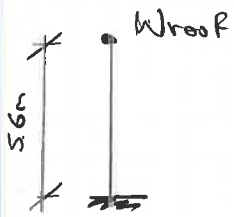
Wind loading: $W_u = 0.80\ kN/m^2$
$W_s = 0.55\ kN/m^2$

Snow loading: $S_u = 0.63\ kN/m^2$
$S_s = 0.25\ kN/m^2$

Seismic loading:

Roof - $G = 0.4 \times 1140 = 456\ kN$
$Q = 0.25 \times 1140 = 285\ kN$
conc panels - $G = 2439.4\ kN$

internal G.b walls - $G = (0.4 \times 2.4) \times (48 + 18 + 21) = 83.5\ kN$



Seismic weight $W_{roof} = 456 + \left(\frac{2439 + 83.5}{2}\right) + 0.3 \times 285$
$W_{roof} = 1347\ kN$

Seismic Shear $V = 1347 \times 0.73 = 983.3\ kN$

$F_{roof} = 983.3\ kN$

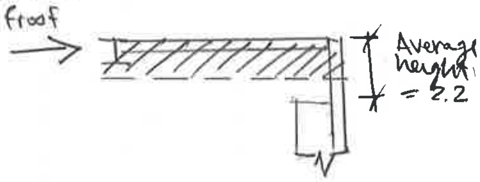
$W_{building} = 1347 + (2439.4 + 83.5/2) = 2608.5\ kN$

Purlins Combined Actions

Max span = 11.6m (Portal + panels Grid 8)

Wall 1 – 35% Seismic Load
Wall 2 – 47% Seismic Load } 82% froof

froof = 983 kN (Conservative)



Concrete panels
G = (2.2/2 x 21 x 3.0 x 4.0) + (2.2/2 x 21 x 3.6 x 1.0) + (2.85/2 x 21 x 3.0 x 1.0)
= 450 kN

Wroof = 450 + (456 x 0.82) + (285 x 0.3 x 0.82) = 894 kN

froof = 894 x 0.73 = 652.6 kN

Purlins @ 1.2m c/c

Axial Load on single purlin $N^* = 653/21 \times 1.2 = 37.3kN$

Wall A – 14% Seismic Load
Wall B – 20% Seismic Load } 34% roof

34% of 652.6 = 222 kN
UDL 222/18 = 12.3 kN/m

Total No. of purlins = 17

UDL on single purlin 12.3/17 = 0.73 kN/m

Lateral Bracing Provided @ 1/3 purlin length
$M_y^* = 0.025wl^2 = 0.3 kNm$
$M_y^* = wl^2/8 = 1.4 kNm$ (Conservative)

Vertical Loads
G = 0.12 kN/m²
Q = 0.25 kN/m²

$W_{G+Q} = 0.44 kN/m$ $W_G = 0.14 kN/m$
$W_{1.2G+1.5Q} = 0.52 kN/m$

200/18 DHS Capacity Load Case G + EQ
$\phi W_{bx} = 0.57 kN/m$
$\phi N_c = 40 kN$
$\phi M_y = 0.9 \times 450 \times 12 \times 10^3 = 4.86 kNm$

$$\frac{N^*}{\phi N_c} + \frac{W^*}{\phi W_b} = \frac{37.3}{40} + \frac{0.14}{0.52} = 1.178$$ 85% NBS

$$\frac{N^*}{\phi N_c} + \frac{M_y^*}{\phi M_y} = \frac{37.3}{40} + \frac{0.3}{4.86} = 0.99$$ 100% NBS

FILE NAME: 52 Hayton Road

DATE: 20.02.14 FILE No: 130329

DESIGNER: PS SHEET No: 03

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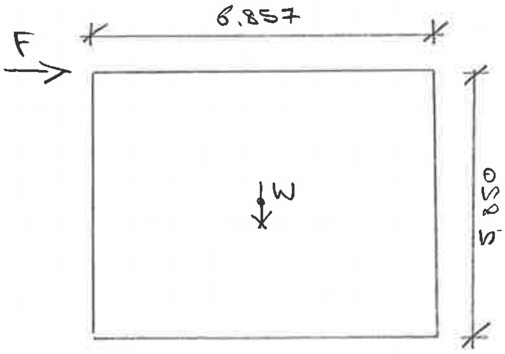
Concrete Panels in plane (North /South Loading)

Grid B

Panel 32

$\mu = 1.25$

$C = 0.73$



EQ Loading:

roof

$W_{roof} = (0.4 \times 9.96 \times 6.857) + (0.3 \times 0.25 \times 9.96 \times 6.857) \times 0.73$

$W_{roof} = 23.7\,kN$

$W_{wall} = 3.0 \times 6.857 \times 5.85 \times 0.5 \times 0.73$

$W_{wall} = 43.9\,kN$

$F = 67.6\,kN$

Overturning Moment

$M_o = 67.6 \times 5.85 = 395.5\,kNm$

restoring Moment

$M_{res} = (3.0 \times 6.857 \times 5.850) \times 6.857/2 = 412.6\,kNm$

$M_{res} \geq M_o$ Ok against overturning $\boxed{100\%\ NBS}$

Shear friction

$V = 67.6\,kN$ $\tan\phi = 0.6$

$V_p = 0.9 \times (3.0 \times 6.857 \times 5.850) \times \tan\phi = 64.98\,kN$

Shear resistance provided by 12mm reid bars cast into R.C columns

4 NO. 12mm reid bars provided per panel edge

Shear capacity reinforcement

$V_s = 0.8 \times 6 \times 6 \times \pi \times 4 \times 300 = 108.6\,kN$

$108.6 + 64.98 = 173.6\,kN \geq 67.6\,kN$

Ok in Shear $\boxed{100\%\ NBS}$

FILE NAME: 52 Hayton Road

DATE: 20.02.14 FILE No: 130329

DESIGNER: PS SHEET No: 04

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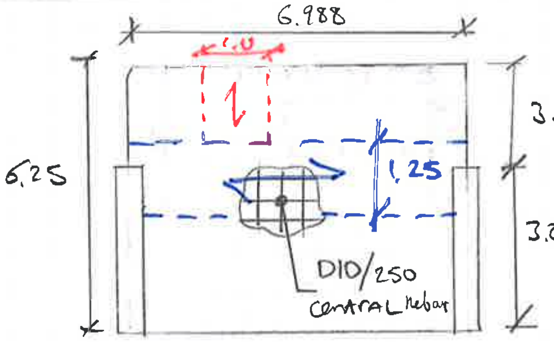
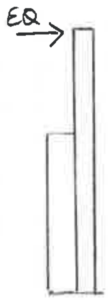
Concrete Panels Face Loading (EAST / WEST LOADING)





Earthquake Loading

Load transferred from roof purlins to top of wall panels

Earthquake Loading

E_{ULS} = 653/21 = 31 kN/m Load resisted by 6 rows of cantilevering wall panels

Load on single wall panel
E_{ULS} = 31 kN/m /6 = 5.2 kN/m

μ = 2.0 E_{ULS} = 5.2 × 0.4 = 2.08 kN/m

1.0m Cantilever Strip
M^* = 2.08 × (3.25 − 1.25/2) = 5.46 kNm

d = 58mm As,prov = 314mm² / metre
f'c = 30 N/mm²
fy = 300 N/mm As,req = 385mm² / metre $\boxed{82\% \text{ NBS}}$

1.25m horizontal spanning strip

M^* = $Wl^2/12$ = $2.08 \times (6.988 - 0.3)^2/12$ = 7.75 kNm

d = 58mm As,prov = 393mm² / 1.25m
f'c = 30 N/mm²
fy = 300 N/mm As,req = 550mm² / 1.25m $\boxed{71\% \text{ NBS}}$

Shear μ = 1.25
V^* = 5.2 × (6.988 − 0.3)/2 = 18 kN
V_s = $A_v f_y d/s$ = 393 × 300 × 58/250 = 27.4 kN

$V_n = V_c + V_s$
$V_c = 0.17\sqrt{f'c}\,A_{cv}$ = $0.83 \times \sqrt{30} \times 1250 \times 125$ = 4.5 kN

V_n = 27.4 + 4.5 = 31.9 kN ok in shear

FILE NAME: 52 hayton road

DATE: 24.02.14 FILE No: 130329

DESIGNER: PS SHEET No: 05

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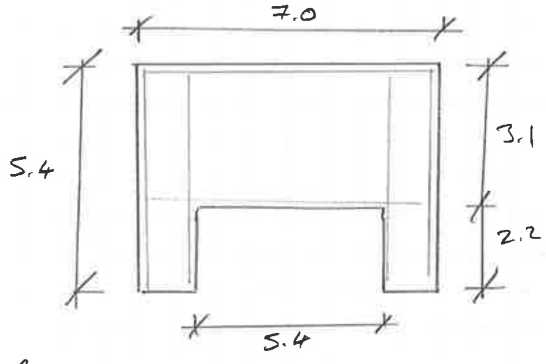
Concrete Panel in-plane loading (East/West loading)

Grid 1 Panel 5/7

EQ loading $\mu = 1.25$
$C = 0.73$



Wroof
$= (0.4 \times 21/2 \times 7.0 + 0.3 \times 0.25 \times 21/2 \times 7.0) \times 0.73$
$= 25.5kN$

Wwall
$= 3.6 \times (7.0 \times 5.5 - 2.2 \times 5.4) \times 0.73$
$= 70kN$

$f = 95.5kN$

Overturning Moment $M_o = 95.5 \times 5.5 = 525.3kNm$

Restoring moment $M_r = \{3.6 \times (7.0 \times 5.5 - 2.2 \times 5.4)/2 \times 0.4\} + \{3.6 \times 26.62/2 \times 6.6\} = 318.6kNm$

$M_r \geq M_o$ ok against overturning $\boxed{100\% \text{ NBS}}$

Shear
$V^* = 95.5kN$

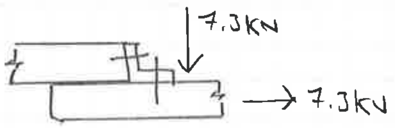
Shear friction $V_p = 0.9 \times 3.6 \times 26.62 \times \tan\alpha = 51.75kN$

reinforcement $V_s = 0.8 \times 5^2 \times \pi \times 6 \times 300 = 113kN$

$V_s > V^*$ ok against shear $\boxed{100\% \text{ NBS}}$

Shear transfered to panels (4+6)/(6+8) + (35)/(41)
through connection 3No. panels
2No. connections per panel

Load on single connection $V^* - V_p = 43.75kN$
$43.75/(3 \times 2) = 7.3kN$



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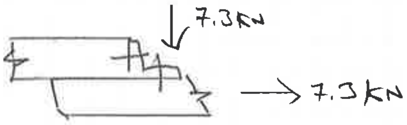
Panel Connection



Safety Bolts B15/45/105
tension Capacity = 37KN
Shear Capacity = 38.88KN

Concrete Strength
$\phi c N t c = 0.6 \times knc\, he f^{1.5} \sqrt{f'c}$
$= 0.6 \times 10 \times 65^{1.5} \sqrt{30} = 17.2KN$

tension
$\phi c \, \xi \, Ntc = 0.6 \times 1.0 \times 1.0 \times 0.75 \times 1.0 \times 17.2 = 7.75KN$

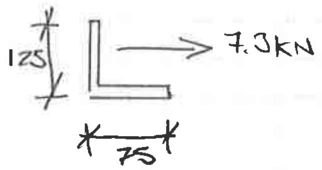
Shear
$\phi c \, Vc = 0.6 \times 1.4 \sqrt{D} \sqrt{f'c} \; e^{1.5} \phi_h$
$= 0.6 \times 1.4 \sqrt{15} \times \sqrt{30} \times 75^{1.5} \times 1.0 = 11.6KN$

Combined tension + shear

$(N^*/\phi c Nc)^2 + (N^*/\phi c Vc)^2 = \left(\frac{7.3}{7.75}\right)^2 + \left(\frac{7.3}{11.6}\right)^2 = 1.3 \leq 1.0$

$\boxed{78\% \ NBS}$

Angle Bracket



10mm thick Bracket in Drawings

Bending of bracket $M^* = 7.3 \times 0.10 = 0.73KNM$

8mm Bracket $Z = Bd^2/6 = 200 \times 8^2/6 = 2133$
$M = 2133 \times 300 = 0.64KNM$

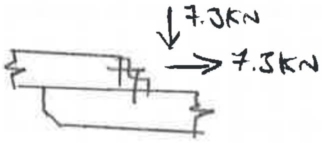
$\frac{M^*}{M} = 1.159 \geq 1.0$ 'FAIL'

$\boxed{86\% \ NBS}$

Shear capacity of bracket $V^* = 7.3KN$
$V = 0.6 \times 300 \times 200 \times 8 = 288KN \geq V^*$

$\boxed{100\% NBS}$

Panel Connection - Threaded insert (TIM 16)



Tension Capacity $\phi N_{uc} = 29.5kN$

$\psi_c = 1.12$
$\phi_{xu} = 0.67$
$\phi_{xy} = 0.77$
$\psi_{cr} = 0.75$

tension Capacity $\phi N_{us} = 29.5 \times 1.12 \times 0.67 \times 0.77 \times 0.75$
$= 12.8kN$

tension Steel Capacity $\phi N_{us} = 33.2kN$

Shear Capacity $\phi V_B = 5.6kN$

$\psi_{vc} = 1.22$
$\psi_5 = 1.0$
$\psi_6 = 1.0$
$\psi_7 = 1.2$

$\phi V_{CB} = 5.6 \times 1.22 \times 1.2 = 8.2kN$

Shear Strength of Steel
$\phi V_s = 23.0kN$

tension Capacity $= 7.3/12.8 = 0.57 \leq 1.0$ OK

Shear Capacity $= 7.3/8.2 = 0.89 \leq 1.0$ OK

Shear + tension
$\frac{N^*}{\phi N} + \frac{V^*}{\phi V} = 0.57 + 0.89 = 1.46 \geq 1.0$ FAIL

$\boxed{68\% \text{ NBS}}$

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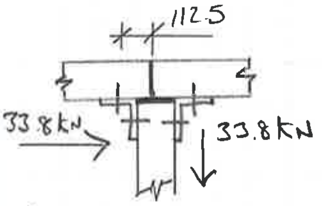
Panel 32 Connection

$f = 67.6\,kN / 2 = 33.8\,kN$

4 Bracket Connections To panels 22 + 23 (2 Top + 2 Centre)



Safety Bolts BS15/45/105
tension Capacity = 37kN
Shear Capacity = 38.85kN

force on single Bracket
$f = 33.8/4 = 8.45\,kN$

Concrete Strength
$\phi_c N_{tc} = 0.6 \times k_{nc}\, h_{ef}^{1.5} \sqrt{f'c} = 0.6 \times 10 \times 65^{1.5} \sqrt{30} = 17.2\,kN$

tension
$\phi_c \Sigma N_{tc} = 0.6 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 17.2 = 10.32\,kN$

Shear
$\phi_c V_c = 0.6 \times 1.4 \sqrt{D} \sqrt{f'c}\; e^{1.5} \psi_h$
$= 0.6 \times 1.4\sqrt{15} \times \sqrt{30} \times 112.5^{1.5} \times 0.79 = 16.8\,kN$

Combined tension + Shear
$(N^*/\phi_c N_c)^2 + (V^*/\phi_c V_c)^2 = \left(\frac{8.45}{10.32}\right)^2 + \left(\frac{8.45}{16.8}\right)^2 = 0.92 \leq 1.0$
OK

$\boxed{100\%\ NBS}$

Angle Bracket

Bending $M^* = 8.45 \times 0.1 = 0.845\,kNM$
Plate Capacity $M = 0.64\,kNM$ (refer to Sheet 6)

$\frac{M^*}{M} = 1.32 \geq 1.0$
'fail'

$\boxed{76\ \%\ NBS}$

Shear $V^* = 8.45\,kN$
Capacity $V = 288\,kN$ (refer to Sheet 7)

$V^*/V = 0.03 \leq 1.0$ OK $\boxed{100\%\ NBS}$

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(North / South Loading)

Concrete Panel Face Loading Grid 1 Panel 5/7

Upper Section - 5.4 x 3.1m

$M = 2.0$ $A_s = 314mm^2/m$

$l = 5.4m$ $b = 1000mm$

$t = 150mm$ $f'c = 30N/mm^2$

$f'y = 300N/mm^2$

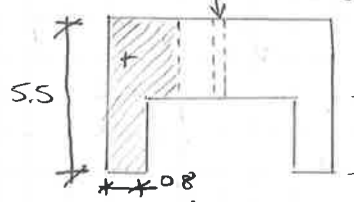
$W = 0.4 \times 0.15 \times 24 = 1.44kN/m$

$M^* = Wl^2/8 = 5.25kNm$

$A_{s,req} = 302mm^2/m$

$\boxed{100\% \text{ NBS}}$

Support provided by return wall



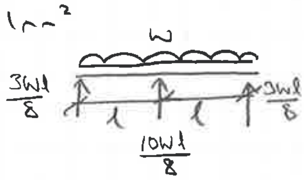
Lower Section - 5.5 x 0.8m

$M = 2.0$ $A_s = 4D10 + 2D12 = 540mm^2$

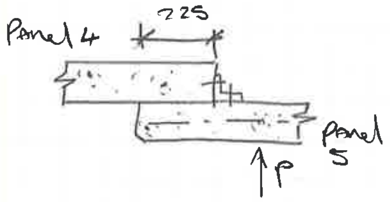
$l = 5.5m$ $b = 800mm$

$t = 150mm$ $f'c = 30N/mm^2$

$f'y = 300N/mm^2$



$P = 1.3Wl/8 = 3.1 \times \frac{1.44 \times 2.7 \times 3}{8} = 4.5kN$



$M^* = 4.5 \times (0.8 - 0.225) = 2.59kNm$

$A_{s,req} = 90mm^2$

$\boxed{100\% \text{ NBS}}$

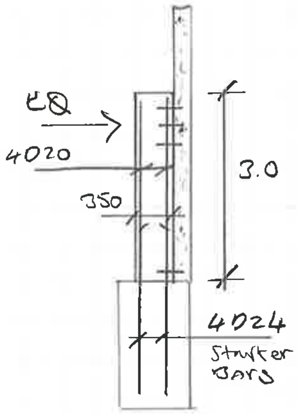
Panel 4 Takes 25% 4.5kN Load
=> 3.375kN Cantilevers

$M^* = 3.375 \times (2.2 + 3.1/2) = 12.66kNm$

$A_{s,req} = 796mm^2$ $540/796 = 0.678$ $\boxed{68\% \text{ NBS}}$

RC Columns

(EAST/WEST LOADING)



Earthquake Loading

$\mu = 2.0$ $f = 67.6$ kN (refer to Sheet 3)
$C = 0.4$

Load acting on R.C Column
$f_x = 67.3 \times 0.4/0.73 = 36.9$ kN

$M^*_x = 2.0 \times 36.9 = 73.8$ kNm

Axial Load:
Col S.W $G = 0.35 \times 0.3 \times 24 \times 3.0 = 7.56$ kN
Wall S.W $G = 0.125 \times 1.2 \times 24 \times 5.85 = 21.06$

$N^* = 28.62$ kN

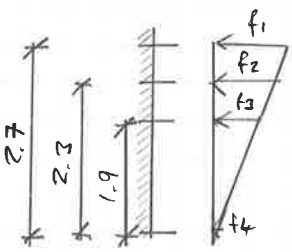
Moment Capacity
$\phi M_x = 51.5$ kNm (D20) $51.5/73.8 = 0.697$ 70% NBS
$\phi M_x = 69.5$ kNm (D24) $69.5/73.8 = 0.94$ 94% NBS

Shear Capacity
$V_s = 0.8 \times 12 \times 12 \times \pi \times 4 \times 300 = 434$ kN

$\mu = 1.25$ $V = 67.3$ kN 100% NBS

Connections

$\mu = 1.25$



$M^* = 67.3 \times 1.5 = 100.95$ kNm

Max tension in Connection
$F_1 = 100.95 \times 2.7 / (1.9^2 + 2.3^2 + 2.7^2) = 16.8$ kN

Reidbar Threaded insert
→ RB12TI
Concrete cone pullout capacity
= 56.9kN

tension capacity RB12 reidbar
$f_y = 56.5$ kN

tension ok 100% NBS

Job number (or name): 130329
Column number: R.C Column

User name : PaulS

Concrete properties:

Rectangular stress block as defined by NZS 3101:1995.
Concrete cylindrical compressive strength = 30.0 MPa
Concrete compression stress coefficient, a1 = 0.85
Compression zone depth coefficient, B1 = 0.85
Concrete maximum strain = 0.0030

Steel properties:

Steel modulus of elasticity = 200 000 MPa
Steel yield strength = 300.0 MPa

Dimensions of the column section:

Rectangular section.
Height of the column section = 350.0 mm
Width of the column section = 300.0 mm
Clear cover to ties parallel to the y-axis = 25.0 mm
Clear cover to ties parallel to the x-axis = 25.0 mm

Results:

Load combination number 1 :
Axial load = 30.7 kN, Mx = 51.5 kNm, My = 0.0 kNm
Strength reduction factor, Phi = 0.85
Required reinforcement ratio = 0.01196, Required reinforcement area = 1255.8 mm2
Initial reinforcement ratio = 0.01196, Initial reinforcement area = 1255.8 mm2
Initial reinforcement ratio scaled by = 1.0000
Moment ratio = 0.00000, Target moment ratio = N/A
Skew angle = 0.0 degrees, NA depth = 43.4 mm
Force (unfactored) carried by concrete = 282.1 kN
Force (unfactored) carried by reinforcement = -246.0 kN
Axial load eccentricity: ex = 0.0 mm, ey = 1677.5 mm

The analysis has been finished.

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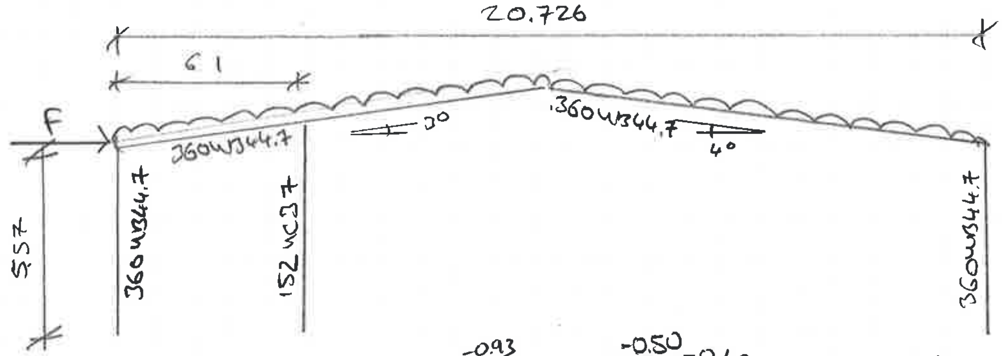
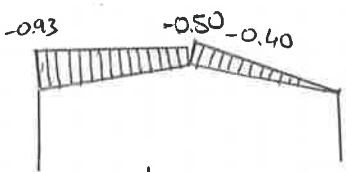
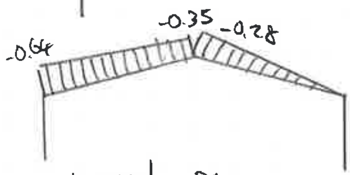
Portal frame



$G = 3.6\,kN/m$
$Q = 2.25\,kN/m$
$S_u = 5.67\,kN/m$
$S_s = 2.25\,kN/m$



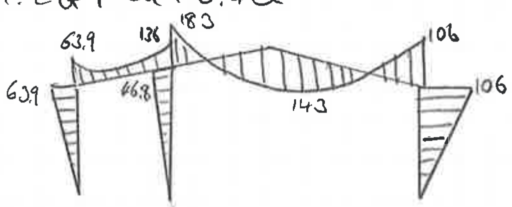


Seismic Loading
Structure S.W $G = 16.1\,kN$

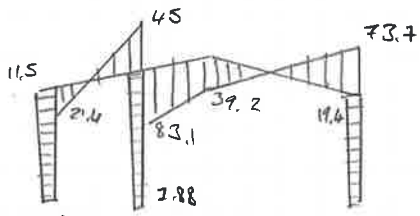
$W_{roof} = (3.6 \times 20.726) + (2.25 \times 20.726 \times 0.3) + 16.1/2 = 96.7\,kN$

$\mu = 2.0$, $f_{ULS} = 96.7 \times 0.40 = 38.7\,kN$
$f_{SLS} = 96.7 \times 0.18 = 17.4\,kN$

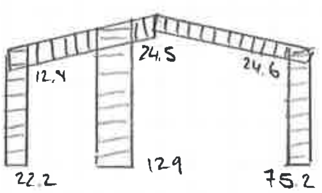
From Microstran Analysis
– worst load case
$1.2G + S_u + 0.4Q$



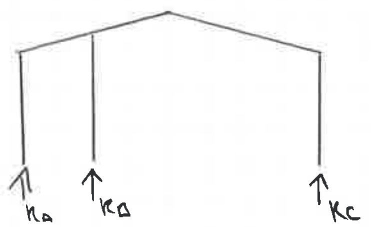
BMD



SFD



Axial forces



Reactions:
$R_A = 11.9\,kN$
$R_B = 69.2\,kN$ } $1.0G + 1.0Q$
$R_C = 40.4\,kN$

Max deflection Y-axis
$\Delta = 51\,mm$ $(1.0G + 1.0Q)$

Max deflection X-axis
$\Delta = 35\,mm$ $(1.0G + 1.0Q)$

FILE NAME: 52 hoyton ROAD

DATE: 24.02.14 FILE No: 130329

DESIGNER: PS SHEET No: 11

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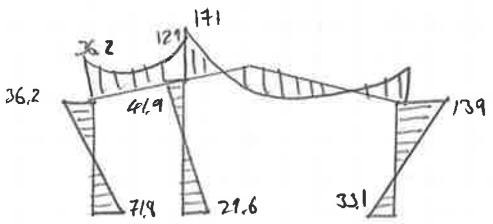
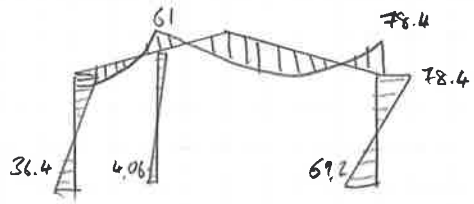
Portal Frame – fixed supports



$1.2G + S_u + 0.4Q$

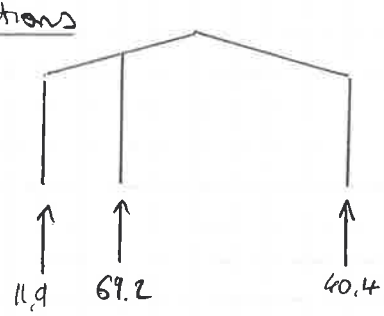
BMD



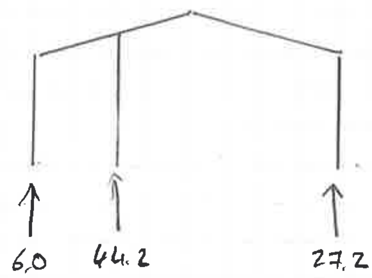
$1.0G + E_{uls} + 0.4Q$

BMD

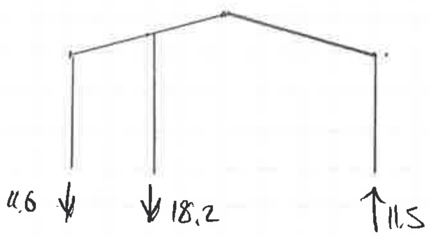
Reactions



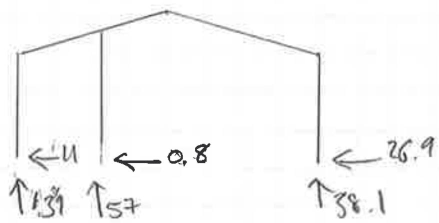
$1.0G + 1.0Q$



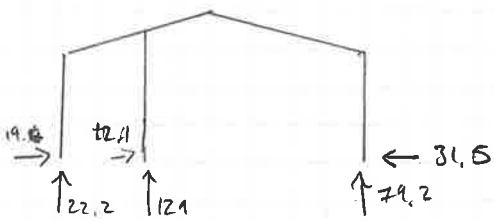
$1.0G + E_s$



$0.9G + W_u$



$1.0G + E_u + 0.4Q$



$1.2G + S_u + 0.4Q$

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Portal Frame - 360UB44.7 Rafters

M* = 183KNM Nc* = 24.6KN
V* = 83.1KN

360UB44.7
ØMsx = 222 KNM ≥ 183KNM Ok
ØVv = 420 KN ≥ 83.1KN Ok

ØNc = 1100KN ≥ 24.6KN ok

deflection Aallow = l/360 = 14.626/360 = 41mm
Δ = 51mm

41/51 = 0.8

Portal Frame - 360 UB 44.7 Stanchions

M* = 110KNM LC 1.0G + EQuls + 0.4Q
V* = 20.8KN LC 1.0G + EQuls + 0.4Q

Nc* = 78.2KN

Lateral support provided by wall cleats @ 2.4m

360UB44.7
ØMsx = 175KNM ≥ 110KNM Ok
ØVv = 420KN ≥ 20.8KN Ok

ØNc = 1514KN ≥ 78.2KN Ok

deflection Aallow = l/150 = 5550/150 = 37mm
Δ = 38mm ≤ 37mm Ok

100% NBS

Portal Frame - 152UC37 Post

M* = 46.8KNM 152UC37
V* = 7.88KN ØMsx = 49.4KNM Le = 5.6m ≥ M* Ok
ØVv = 220KN ≥ V* Ok
Nc* = 129KN ØNc = 780 KN ≥ Nc* Ok

deflection Aallow = l/150 = 37mm
Δ = 35mm ≤ 37mm Ok

100% NBS

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foundations

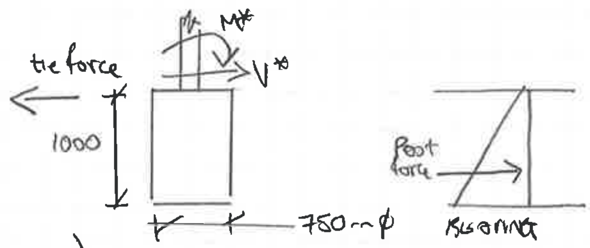
Post hole @ Portal frame

Drawing sheet 2 foundation plan → Post hole C 750mm ϕ

Drawing sheet S1 Portal frame details → 900mm ϕ posthole

Check smaller 750mm ϕ posthole

From Microstran Analysis



$M^* = 71.8\ kNm$
$V^* = 31.5\ kN$

tie force $= \left(\frac{3 \times 31.5}{2 \times 1.0}\right) \times \left(\frac{71.8}{31.5} + \frac{2}{3} \times 1.0\right)$

tie force $= 139.2\ kN$

Post force $= 71.8 / (2/3 \times 1.0) = 107.7\ kN$

BEARING $= 2 \times 107.7 / (1.0 \times 0.75) = 287.2\ kPa$

tie reinforcement

tie force provided by 2NO. D20 Column ties

tie $= 2 \times \pi \times 10^2 \times 300 \times 0.9 = 169\ kN \geq$ tie force $= 139.2\ kN$

$\boxed{100\%\ NBS}$

Post hole reinforcement

Main reinforcement 4D24 $A_{s,prov} = 1809\ mm^2$

$M^* = 71.8\ kNm$
$A_{s,req} = 414\ mm^2$ Sufficient reinforcement provided

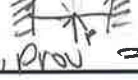
$\boxed{100\%\ NBS}$

Resistance to uplift force

uplift force $= 18.2\ kN$

Resistance to uplift $= (0.75/2)^2 \pi \times 1.0 \times 24 = 10.6\ kN$

Utilize weight of 125mm SLAB ; Area req $= 18.2 - 10.6 / (0.125 \times 24)$
$= 2.53\ m^2$

1.6 x 1.6 Area of SLAB  $M^* = Pl/8 = 1.48\ kNm$

$A_{s,req} = 90\ mm^2$ $A_{s,prov} = 1NO.\ 12mm$ Bar $= 113\ mm^2$ Ok

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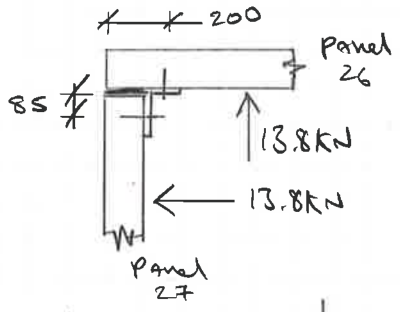
Connections

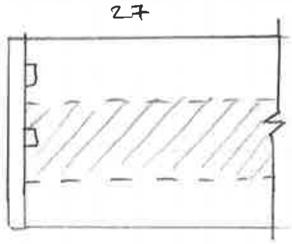
Connection between Panel 27 + 26



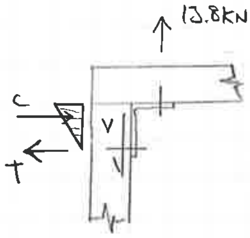
Panel 26 S.W = 113.8KN
Panel 27 S.W = 91.5KN

Seismic Loading $\mu = 1.25$ $C = 0.73$

Panel 26
$0.73 \times 113.8/3 \times 0.5 = 13.8\,KN$



Panel 27
$(0.73 \times 91.5/3 \times 0.5) + (0.4 \times 5.0 \times 6.25/2 \times 0.73) \times 0.5$
$+ (0.3 \times 0.25 \times 5.0 \times 6.25/2 \times 0.73) \times 0.5$
$= 13.8\,KN$



$V^* = 13.8\,KN$
$N_t^* = \dfrac{13.8 \times 0.05}{100(2/3)}$
$N_t^* = 10.4\,KN$

Safety Bolts B15/45/105
tension Capacity = 37KN
Shear Capacity = 38.85KN

Concrete Strength
$\phi_c N_{tc} = 0.6 \times k_{nc}\, h_{ef}^{1.5} \sqrt{f'c} = 0.6 \times 10 \times 65^{1.5} \sqrt{30} = 17.2\,kN$

tension
$\phi_c \, \Sigma N_{tc} = 1.0 \times 1.0 \times 1.0 \times (0.3 + 0.7 \times 85/1.5 \times 65) \times 17.2 \times 0.6 = 9.39\,kN$

$\dfrac{9.39}{10.4} = 0.9$ $\boxed{90\%\ NBS}$

Shear
$\phi_c V_c = 1.4 \sqrt{D} \sqrt{f'c}\ e^{1.5} \psi h$
$= 1.4 \sqrt{15} \sqrt{30}\ 85^{1.5} \times 1.0 = 23.3\,kN$
'ok in Shear'

$\boxed{100\%\ NBS}$

Combined Shear + tension
$(N^*/\phi_c N_c)^2 + (V^*/\phi_c V_c)^2 = 1.58 \geq 1.0$
'FAIL in Shear + tension'

$\boxed{63\%\ NBS}$

Angle Bracket

$M^* = 13.8 \times 0.05 = 0.69\,kNM$
Capacity $M = 0.29\,kNM$

$M^*/M = 2.16 \geq 1.0$ 'FAIL'

$\boxed{40\%\ NBS}$

Shear $V^* = 13.8\,KN$
Capacity $V = 288\,KN$

$V^*/V = 0.05 \leq 1.0$ OK $\boxed{100\%\ NBS}$

Appendix 4

Standardised Report Form

Detailed Engineering Evaluation Summary Data V1.14

Location

Building Name:	52 Hayton Road	Reviewer:	Matthew Blyth
Building Address (Unit / No: / Street):	A - E / 52 / Hayton Road	CPEng No:	237435
Legal Description:		Company:	TM Consultants
		Company project number:	130699
		Company phone number:	03 348 6066
GPS south (Degrees / Min / Sec):	43 / 32 / 32.30	Date of submission:	
GPS east (Degrees / Min / Sec):	172 / 33 / 25.90	Inspection Date:	19.08.11
		Revision:	A
Building Unique Identifier (CCC):		Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	0
Soil type:	sandy silt	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	22.00

Building

No. of storeys above ground:	1	single storey = 1	
		Ground floor elevation (Absolute) (m):	22.20
Ground floor split?	no	Ground floor elevation above ground (m):	0.20
Storeys below ground	0		
Foundation type:	other (describe)	if Foundation type is other, describe:	Post Hole Foundations
Building height (m):	6.50	height from ground to level of uppermost seismic mass (for IEP only) (m):	5.5
Floor footprint area (approx):	1140		
Age of Building (years):	16	Date of design:	1992-2004
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	public	Brief strengthening description:	
Use (upper floors):	other (specify)		
Use notes (if required):	Industiral		
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	load bearing walls		
Roof:	other (note)	describe system	Zincalume Trimdek cladding on 200/18 DHS purlins at 1.2m c/c.
Floors:	concrete flat slab	slab thickness (mm)	125
Beams:	steel non-composite	beam and connector type	360UB Portal Frame, moment resisting connections
Columns:	cast-insitu concrete	typical dimensions (mm x mm)	350 x 300
Walls:	load bearing concrete	#N/A	

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	single level tilt panel		note total length of wall at ground (m):	48
Ductility assumed, μ:	1.25			
Period along:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	single level tilt panel		note total length of wall at ground (m):	21
Ductility assumed, μ:	1.25			
Period across:	0.14	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

north (mm):		leave blank if not relevant
east (mm):		
south (mm):		
west (mm):		

Non-structural elements

Stairs:	timber	describe supports	Stringers supported at ground & 1st floor (Unit A)
Wall cladding:	precast panels	thickness and fixing type	125mm / 150mm precast concrete panels
Roof Cladding:	Metal	describe	Zincalume Trimdek roof cladding
Glazing:	aluminium frames		
Ceilings:	none		
Services(list):			

Available documentation

Architectural	partial	original designer name/date	Chris Bunn Design, 1998
Structural	partial	original designer name/date	Engineering & Building Advisory Service, 1998
Mechanical	none	original designer name/date	-
Electrical	none	original designer name/date	-
Geotech report	none	original designer name/date	-

Damage

Site:
(refer DEE Table 4-2)

Site performance:	Good	Describe damage:	Minor cracking to concrete panels & internal Gib partitions. Damage to a number of concrete panel connections.
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	slight	notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	No Structural Damage
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

$$Damage\_Ratio = \frac{(\%NBS(before) - \%NBS(after))}{\%NBS(before)}$$

Recommendations

	Level of repair/strengthening required:	minor structural		Describe:	Additional panel connection brackets installed to increase the panel connections to 67% NBS
	Building Consent required:	no			
	Interim occupancy recommendations:	full occupancy			
Along	Assessed %NBS before e'quakes:	67%	0% %NBS from IEP below	If IEP not used, please detail assessment methodology:	Detailed analysis undertaken
	Assessed %NBS after e'quakes:	67%			
Across	Assessed %NBS before e'quakes:	67%	0% %NBS from IEP below		
	Assessed %NBS after e'quakes:	67%			

IEP

Use of this method is not mandatory - more detailed analysis may give a different answer, which would take precedence. Do not fill in fields if not using IEP.

Period of design of building (from above): 1992-2004

h_n from above: 5.5m

Seismic Zone, if designed between 1965 and 1992:

not required for this age of building

Design Soil type from NZS4203:1992, cl 4.6.2.2:

	along	across
Period (from above):	0.4	0.139
(%NBS)nom from Fig 3.3:		

Note:1 for specifically design public buildings, to the code of the day: pre-1965 = 1.25; 1965-1976, Zone A =1.33; 1965-1976, Zone B = 1.2; all else 1.0	1.00
Note 2: for RC buildings designed between 1976-1984, use 1.2	1.0
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)	1.0

	along	across
Final $(\%NBS)_{nom}$:	**0%**	**0%**

2.2 Near Fault Scaling Factor

Near Fault scaling factor, from NZS1170.5, cl 3.1.6: 1.00

	along	across
Near Fault scaling factor (1/N(T,D), **Factor A**:	1	1

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:

Z_{1992}, from NZS4203:1992

Hazard scaling factor, **Factor B:** #DIV/0!

2.4 Return Period Scaling Factor

Building Importance level (from above): 2

Return Period Scaling factor from Table 3.1, **Factor C**:

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)	1.00	1.00
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	1.00	1.00

2.6 Structural Performance Scaling Factor:

	along	across
Sp:	1.000	1.000
Structural Performance Scaling Factor **Factor E**:	1	1

2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom} \times A \times B \times C \times D \times E$ **$\%NBS_b$:**

Global Critical Structural Weaknesses: (refer to NZSEE IEP Table 3.4)

3.1. Plan Irregularity, factor A: 1

3.2. Vertical irregularity, Factor B: 1

3.3. Short columns, Factor C: 1

3.4. Pounding potential

Pounding effect D1, from Table to right 1.0

Height Difference effect D2, from Table to right 1.0

Therefore, Factor D: 1

3.5. Site Characteristics 1

Table for selection of D1	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Alignment of floors within 20% of H	**0.7**	**0.8**	**1**
Alignment of floors not within 20% of H	**0.4**	**0.7**	**0.8**

Table for Selection of D2	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Height difference > 4 storeys	0.4	0.7	1
Height difference 2 to 4 storeys	0.7	0.9	1
Height difference < 2 storeys	1	1	1

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum		
Rationale for choice of F factor, if not 1		

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)

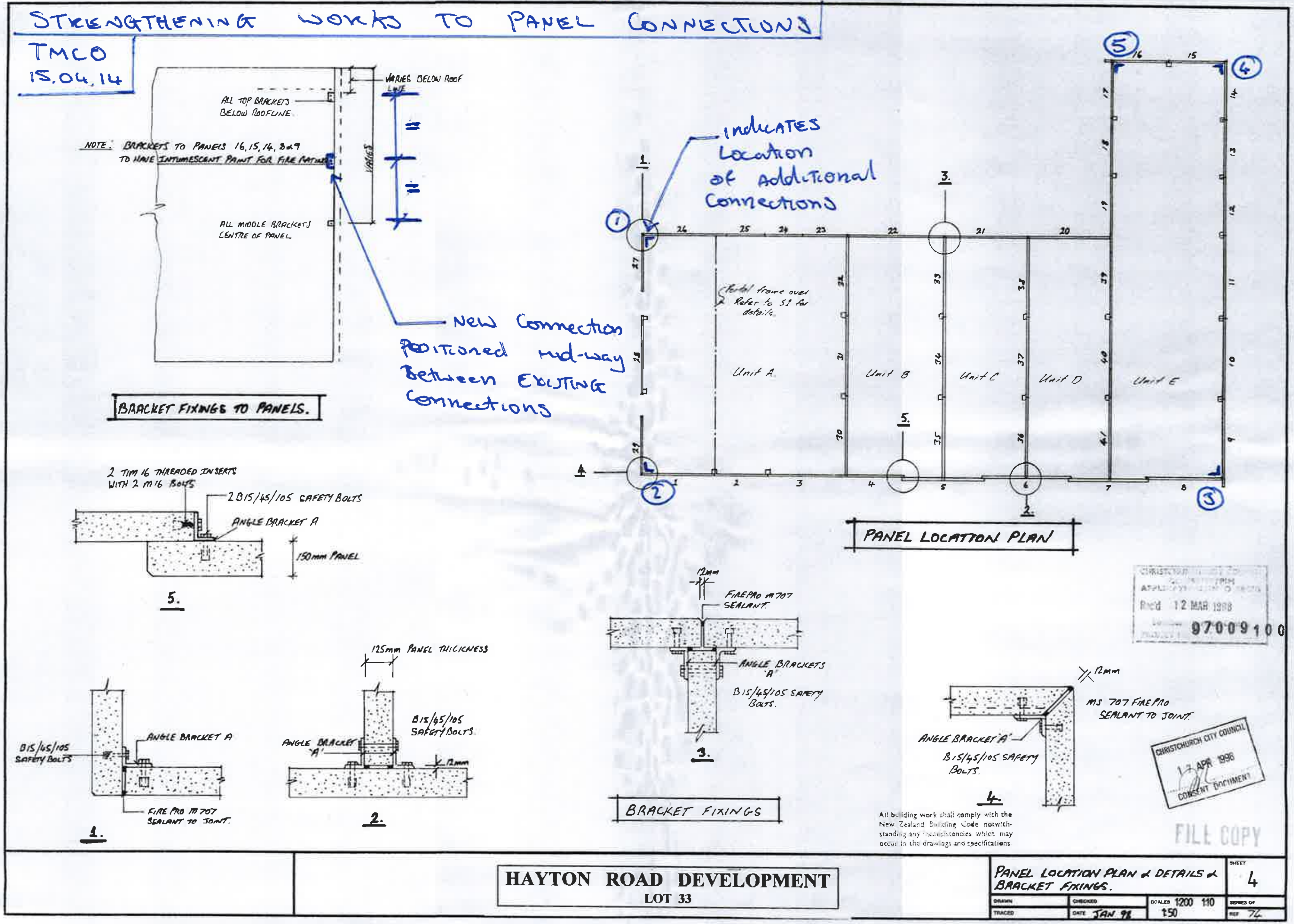
List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

	Along	Across
3.7. Overall Performance Achievement ratio (PAR)	**0.00**	**0.00**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:		
4.4 Percentage New Building Standard (%NBS), (before)		

Appendix 5

Strengthening Work to Upgrade Building to 67% NBS

52 HAYTON ROAD JOB NO. 130329



FILE NAME: 52 Hayton Road

DATE: 15.04.14 FILE No: 130329

DESIGNER: PS SHEET No: 01

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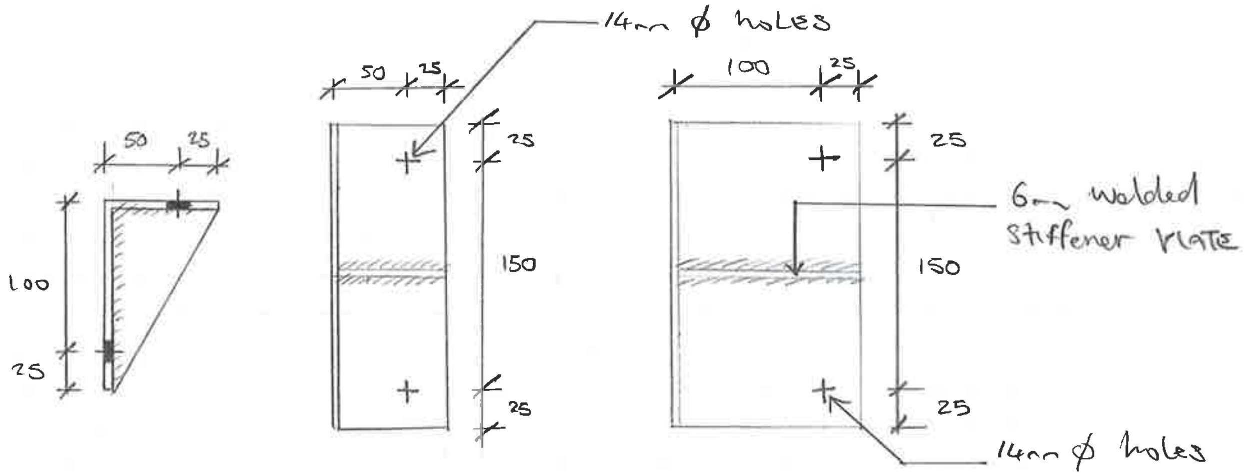
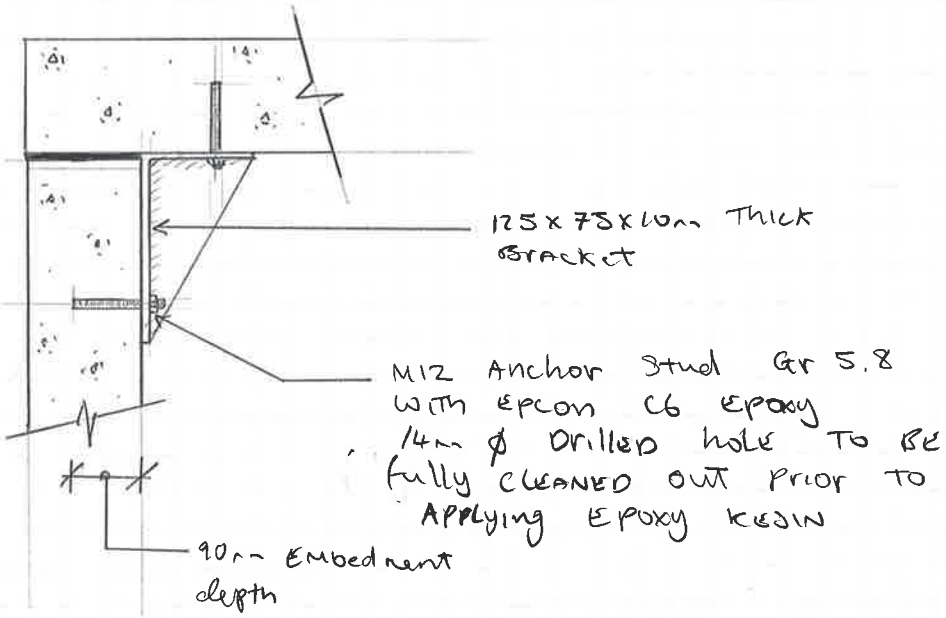


New 125 x 75 x 10mm Thick Brackets 200mm long
(Total No. required = 5)



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FILE NAME: 52 Hayton Road

DATE: 16.04.14 FILE No: 130329

DESIGNER: PS SHEET No: 01

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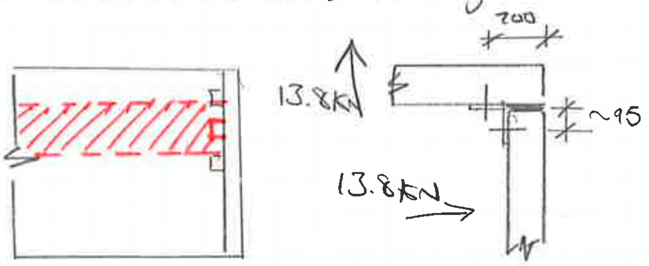
Connection Strengthening To Achieve 67% NBS



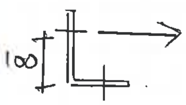
Capacity of EXISTING Connections:

- Anchors tension = 9.39KN (90% NBS)
- Anchors Shear = 23.3KN (100% NBS)
- Anchors tension + Shear 1.58 ≥ 1.0 (63% NBS)

- Bracket Capacity
  $\phi M = 0.9 \times 320 \times 100 \times 10^2/6 = 0.48 KNM$ (35% NBS)

- Bracket Shear Capacity
  $\phi V = 0.9 \times 320 \times (100 - 22) \times 10 = 225 KN$ (100% NBS)

New Connection



(100mm long Bracket)

67% NBS	100% NBS
$F = ((13.8\times2)-(4.8\times2))\times0.67$	$F = (13.8\times2)-(4.8\times2)$
$F = 12.1KN$	$F = 18KN$
$M^* = 1.21KNM$	$M^* = 1.8KNM$
Bracket Thickness	Bracket thickness
$t = \sqrt{\frac{(1.21\times10^6)\times6}{0.9\times320\times100}}$	$t = \sqrt{\frac{1.8\times10^6\times6}{0.9\times320\times100}}$
$t = 16mm$	$t = 20mm$

Provide 16mm Thick Angle Bracket (Conservative)

- Capacity of Top Bracket
  $\phi M = 0.9 \times 320 \times 200 \times 10^2/6 = 0.96 KNM$ (70% NBS)

$F = \{(13.8\times2)-(4.83+9.66)\}\times0.67 = 8.8KN$ $M^* = 0.9KNM$

Bracket thickness $= \sqrt{\frac{0.9\times10^6\times6}{0.9\times320\times100}} = 14mm$

Using 12mm Bracket Require length $= \frac{0.9\times10^6\times6}{0.9\times320\times12^2} = 130mm$

FILE NAME: 52 hayton ROAD

DATE: 15.04.14 FILE No: 130329

DESIGNER: PS SHEET No: 02

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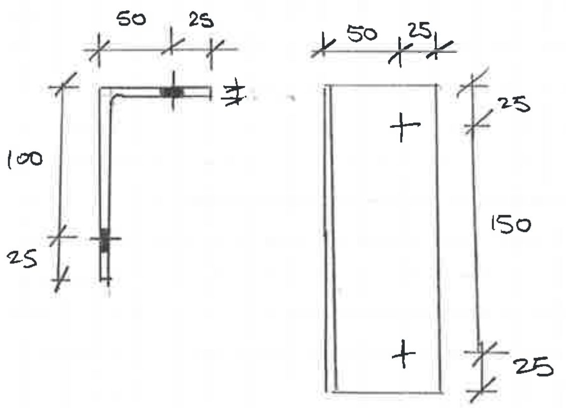
Provide 125 x 75 x 12 unequal Angle 200mm long
or 125 x 75 x 10 unequal angle 200mm long



Bracket Capacity:

$\phi M = 0.9 \times 300 \times 200 \times 12^2/6$

$\phi M = 1.296$ kNm

$\phi V = 0.9 \times 300 \times 200 \times 12$

$\phi V = 648$ kN

Anchor Capacity: (single Anchor) M12 Anchor Stud Gr 5.8 Epcon C6

tension $\phi N_{ur} = 23.0$ kN

Shear $\phi V_{ur} = 10.3$ kN

Combined Shear + tension

$N^*/\phi N_{ur} + V^*/\phi V_{ur} = 0.98 \leq 1.0$

Applied force = 8.8 kN

$M^* = 0.88$ kNm

$V^* = 8.8$ kN

Connection achieves $\boxed{147\% \text{ NBS}}$ (12mm Bracket)

using 10mm angle bracket → $\phi M = 0.9$ kNm

$\phi V = 560$ kN

Connection achieves $\boxed{102\% \text{ NBS}}$ (10mm Bracket)

Chemical Anchoring - ChemSet Anchor Stud Design Calculator

Ramset™

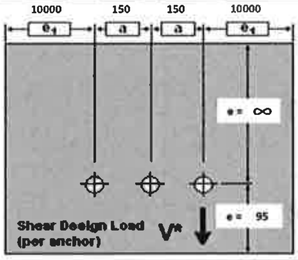
Injection Sys: Epcon™ C6 Series

Anchor Type: Anchor Stud - Gr 5.8

Anchor Size d_b = M 12

Input Description (Strength Limit State Design)	Input Data (per anchor)		
1. Number of anchors (n)	n =	2	-
2. Anchor Spacing (a)	a =	150	mm
3. Concrete Edge Distance (e)	e =	95	mm
4. Concrete Cylinder Strength (f'c)	f'c =	25	MPa
5. Concrete Cube Strength (β)	β =	31	MPa
6. Effective Depth (h>6xdh)	h =	90	mm
7. Anchor Stud size (d_b) - M8 → M24	d_b =	12	mm
8. Concrete Edge Distance Corner (e1)	e_1 =	10000	mm
9. Internal to a row (I) or end of row (E)	I or E	I	row
10. Dry Hole (D) or Wet hole (W)	D or W	D	-
11. Min Concrete Sub'te Thickness (b_m)	b_m =	118	mm
12. Anchor Stud Grade (5.8, 8.8, 316 SS)	Grade =	5.8	Gr
13. Fixture Thickness (t)	t =	20	mm
14. Effective Length (L_e)	L_e =	110	mm
15. Design Tensile Load-per anchor (N*)	N* =	7	kN
16. Design Shear Load-per anchor (V*)	V* =	7	kN
17. Direction of Shear design load (α)	α =	0	°

Plan View - Generic
Dimensions in (mm)



Project Details
Project Name:-
52 Hayton Road
Project Site Address:-
Company Name:-
TMCO
Design Identification:-
Additional Panel Connections
Date:-
14.04.14

ChemSet™ Anchor Stud "Specification"

Part No. CS12160 or CS12160GH

Hole Diameters (mm)	Capacity Reduction Factors
Drill d_h = 14	Conc. Tension/Shear ϕ = 0.6
Fixture d_f = 15	Steel Tension/Shear ϕ = 0.8

Epcon™ C6 Series "Specification"

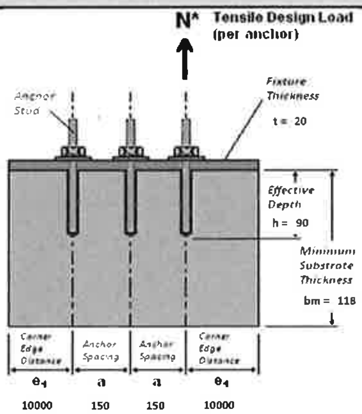
C6-18

Direction of Shear Design Load - α

α = 0° - towards edge
α = 180° - away from edge

Output Description (Strength Limit State Design)	Output Data (per anchor)		
DESIGN O.K.	MIN. CRITERIA for a,e & h - O.K.		
Des. Red. Ult. CONC. Tensile Capacity	ϕN_{urc} =	23.0	kN
Red. Char. Ult. STEEL Tensile Capacity	ϕN_{us} =	33.8	kN
Des. Red. Ult. CONC. Shear Capacity	ϕV_{urc} =	10.3	kN
Red. Char. Ult. STEEL Shear Capacity	ϕV_{us} =	21.0	kN
Drill hole diameter	d_h =	14	mm
TENSION O.K.			
Design Red. Ult. Tensile Capacity	**ϕN_{ur} =**	**23.0**	**kN**
Tension Design Check	**$N^*/\phi N_{ur}$ =**	**0.30**	**< 1**
SHEAR O.K.			
Design Red. Ult. Shear Capacity	**ϕV_{ur} =**	**10.3**	**kN**
Shear Design Check	**$V^*/\phi V_{ur}$ =**	**0.68**	**< 1**
COMBINED TENSION SHEAR O.K.			
Combined Check - $N^*/\phi N_{ur} + V^*/\phi V_{ur}$ =		**0.98**	**< 1.2**

Elevation View - Generic
Dimensions in (mm)



Anchor Loaded

"E" Anchor end of a row

"I" Anchor internal to a row

Anchor Size	d_b	Metric	8	10	12	16	20	24	30	36
Drill hole diameter	d_h	(mm)	10	12	14	18	24	26	N/A	N/A
Stressed Area	A_b	(mm²)	33	53	79	154	232	337	N/A	N/A
Anchor Stud Yield Strength	f_y	(MPa)	430	430	430	420	420	420	N/A	N/A
Red.Char. Ult. Steel Tensile Capacity	ϕN_{us}	(kN)	14.3	22.7	33.8	64.7	97.6	141.3	N/A	N/A
Red.Char. Ult. Steel Shear capacity	ϕV_{us}	(kN)	8.9	14.1	21.0	39.7	59.9	86.8	N/A	N/A
Edge distance for no conc.cone reduction	e_c	**(mm)**	**35**	**40**	**50**	**65**	**80**	**100**	N/A	N/A
Anchor spacing for no conc.cone reduction	a_c	**(mm)**	**50**	**60**	**75**	**100**	**120**	**145**	N/A	N/A
Absolute Minimum edge dist. & anc'r spac.	e_m & a_m	**(mm)**	**25**	30	**35**	**50**	**60**	**75**	N/A	N/A

Effective Depth - h		**Design Reduced Ultimate tensile capacity ϕN_{ur} (kN per anchor) Based on edge distance (e_c) and anchor spacing (a_c) for no conc.cone reduction**							
60	(mm)	10.6							
65	(mm)	12.0							
70	(mm)	13.4	14.6						
80	(mm)	14.3	17.9						
90	(mm)	14.3	21.4	23.0					
100	(mm)	14.3	22.7	26.9					
110	(mm)	14.3	22.7	31.0	34.8				
125	(mm)	14.3	22.7	33.8	42.2				
140	(mm)	14.3	22.7	33.8	50.0				
150	(mm)	14.3	22.7	33.8	55.5	60.7			
160	(mm)	14.3	22.7	33.8	61.1	66.8	71.9		
170	(mm)	14.3	22.7	33.8	64.7	73.2	78.7		
180	(mm)	14.3	22.7	33.8	64.7	79.7	85.8		
190	(mm)	14.3	22.7	33.8	64.7	86.5	93.0	N/A	
220	(mm)	14.3	22.7	33.8	64.7	97.6	115.9	N/A	
240	(mm)	14.3	22.7	33.8	64.7	97.6	132.1	N/A	N/A
270	(mm)	14.3	22.7	33.8	64.7	97.6	141.3	N/A	N/A
330	(mm)	14.3	22.7	33.8	64.7	97.6	141.3	N/A	N/A
350	(mm)	14.3	22.7	33.8	64.7	97.6	141.3	N/A	N/A

The design engineer should ensure the structural element is capable of supporting these loads. Refer to Ramset™ Specifiers Anchoring Resource Book Version 5 for more information or explanation of technical data.

TM CONSULTANTS
KNOWLEDGE YOU CAN BUILD ON

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To: Chris Bunn

SR No: 01

File No: 130329

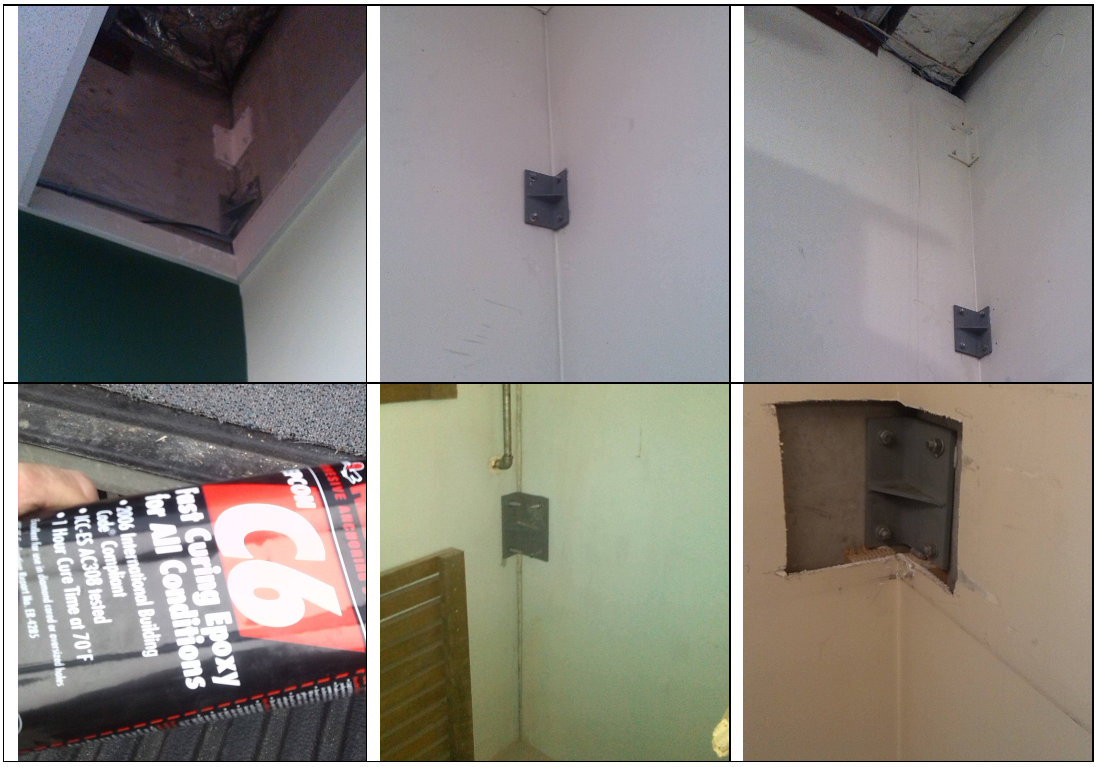
Date: 13.05.14

Project: 52 Hayton Road

SITE REPORT

Inspection of Strengthening Work Undertaken at 52 Hayton Road to Upgrade Building to 67% NBS

- All 5 additional brackets installed as per TMCO sketches dated 15.04.14.
- Bracket size & dimensions as per TMCO sketches.
- Brackets fixed with Epcon C6 epoxy as per TMCO sketches.



Following the installation & fixing of the additional brackets, the building at 52 Hayton Road achieves 67% NBS.

Signature on behalf of TM Consultants Limited: Paul Sweeney