HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS

STRUCTURAL REPORT

OFFICE BUILDING
249 MADRAS STREET

Prepared for

CANTERBURY REGIONAL COUNCIL

by Holmes Consulting Group, Christchurch

in association with Buddle Findlay Limited
and Schulz Knight Consultants Limited

January 1990

Holmes Consulting Group Limited,
61 Cambridge Terrace,
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Offices in Christchurch, Wellington, New Plymouth, Auckland.

CONTENTS

1.0 INTRODUCTION

Holmes Consulting Group Limited were engaged on 24th January 1990 by Buddle Findlay Limited and Schulz Knight Consultants Limited to prepare a structural report on the office development located at 249 Madras Street. The building was completed during 1987 and is currently untenanted.

2.0 PEOPLE INVOLVED WITH CONSTRUCTION OF THIS BUILDING

Developer	Prime West Corporation
Contractor	Williams Construction Limited
Architect	Alun Wilkie Architects
Structural Engineer	Alan M. Reay Consulting Engineer
Mechanical Consultant	
Electrical Consultant	
Soils Consultant	Soils & Foundations Limited

3.0 CONCLUSIONS

Due to the limited time available for the report, our review has been limited to a brief inspection of the building and documents, and approximate calculations. No materials testing has been undertaken, and inspection has been limited to such areas as were readily accessible. Given these qualifications, our conclusions are as follows:-

1. The building is in a condition appropriate to its age and the contractor-as-developer form of construction.

2. The layout and design of the building is quite simple and straight forward and generally complies with current design loading and materials codes.

3. A vital area of non-compliance with current design codes, seen in the documents, is in the tying of the floors to some of the shear walls. This item is under review with the original consultants, but if confirmed will require potentially expensive remedial work. However, this cost is a matter for discussion between the current owner and their consultants.

4. Apart from ongoing maintenance costs which should be minor, no major costs are anticipated in association with the structure, subject to 3. above.

4.0 SUMMARY OF INVESTIGATION

A full set of Architectural drawings, and some structural drawings were made available from Alun Wilkie Architects.

In addition, we were able to view the full design, documentation, Soils Investigation and complete set of drawings at the office of Alan M. Reay Consulting Engineer, on 26 January 1990. The original design engineer was unavailable for comment, having since left the company, but Mr Geoff Banks was available for comment on aspects of the design.

We have spoken to Mr Bryan Bluck, Buildings Control Manager at the Christchurch City Council, to discuss any concerns relating to the building permit and construction process.

An inspection was made on 30th January 1990. Levels 1 and 4 were unavailable for inspection, but the remaining floors were taken as representative. Access was gained to the Lift Machine room, Cooling tower and onto the roof.

5.0 DESCRIPTION

1.	No. storeys and occupancy:	5 storeys office (floor to floor height typically 2600 clear) and ground floor parking.
2.	Gross Floor dimensions:	approx. 31m x 22.5 m.
3.	Foundation type:	Shallow strip footings and foundations pads, with large foundation walls under structural shear walls.
4.	Suspended Floors:	200mm overall insitu concrete on metal tray, supported by precast concrete beams on insitu columns on a 7.5m x 7.0m grid generally.
5.	Roof construction:	Lightweight metal cladding on steel purlins and beams, supported on insitu concrete columns.
6.	Floor Design liveloads:	2.5 kPa typically (minimum load level required by NZS 4203 : 1984).
7.	Lateral load resistance:	This is via a reinforced concrete coupled shear wall on the south face of the building, and a system of reinforced concrete walls around the service core on the north face of the building.
8.	Exterior Cladding:	400 deep x 100 mm precast spandrel panels with glazing between, or on West elevations 140 mm blockwall to level 4 with metal cladding above perforated for windows.
9.	Exterior maintenance:	No allowance for a Building Maintenance Unit has been made. Access for external cleaning is through windows. With opening windows restricted to a single pair approx. 1.0 m wide per 7.5 m bay, this is limited, although the spandrel panels are sufficiently wide for a person to stand safely.

6.0 STRUCTURAL DESIGN ASPECTS

6.1 Foundations

From the soils investigation report prepared by Soils and Foundations Limited, we note that settlement was highlighted as a potential problem, particularly in the north-east corner of the site, causing differential settlement concerns. The pad and strip foundations were sized using the recommendations of the report on maximum allowable stresses. However the recommendations of the report on a maximum pressure to limit settlement appear not to have been followed. It is not known whether any ground improvement work was undertaken to compensate for this.

However, inspection of the site revealed no sign of any significant settlement. Given that most settlement occurs within a relatively short time of construction, this should not become a significant problem in the future.

6.1 Gravity Structure

From our perusal of the drawings, and our investigation of the building, it appears the gravity structure is sound and complies in all respects with the appropriate design loading and materials codes. Furthermore it was noted in the documentation that although only a 2.5 kPa standard office live load was called for, the floor will withstand a live load of up to 3.4 kPa. This would be subject to further confirmation.

6.3 Lateral load resistance

Resistance to lateral loads is via reinforced concrete shear walls.

The shear walls themselves appear to have been generally well designed to the requirements of the correct design loading and materials codes. The building was apparently analysed using a 3 dimensional computer analysis programme checked by a static hand analysis.

An area of concern however has been discovered in the connections of the structural floor diaphragm to the shear walls. While this is not a concern on the coupled shear wall to the south of the building, connections to the walls at the North face of the building are tenuous, due to penetrations for services, lift shafts and the stairs, as detailed on the drawings.

The result of this would be that in the event of an earthquake, the building would effectively separate from the shear walls well before the shear walls themselves reach their full design strength.

Discussion has continued on this matter with Mr Geoff Banks of Alan Reay Consulting Engineer , and it currently appears that there may have been some provision made for this during construction. However, no documentation apparently exists, so it would only be safe to assume that this aspect fails to comply with current design codes.

6.4 Roof

Due to its light weight nature, the roof is prone to deflections, particularly in wind. A brief check shows that the deflections should be within allowable limits, as prescribed in the current codes. However, in our experience, movement may be quite perceptible and disconcerting for the occupants and in extreme wind, may cause damage to ceiling tiles.

Furthermore, it was noted on inspection that the internal butynol lined gutters at roof level have only one downpipe with no provision for an overflow. This is a potential problem in the event of a blockage to a downpipe.

6.5 Fire Escape

On the south face there is a steel cantilevering fire escape. This is currently in good condition but it should be noted that this type of construction is prone to corrosion and should be the subject of an on-going maintenance programme.

7.0 CONDITION REPORT

As expected for a building of this age, the structure appears generally in sound condition. Although mainly concealed by carpets and ceilings, those parts of the structure accessible to view reveal no signs of distress.

Standards of workmanship are adequate although finishes and details appear to have been given the minimum of effort. This is commensurate with the type of development and the time at which it was built.

There has been some water damage to ceiling tiles at level 5 adjacent to the wall between the lifts and the stairwell. This is probably due to a failed flashing.

During the inspection it was noted that there is evidence of cracking on the end of the spandrel panels on either side of the fire escape. The finish in these areas is different to the rest of the panels. It appears that the crack has formed at the interface between the spandrel panel itself and the beam supporting it. In the worst instance this crack may propagate above floor level and cause waterproofing problems.

The roof is mainly in good condition, although several panels of the Trimdek roofing have been dented quite badly. Furthermore, there is evidence of some ponding in the gutters which appear to have minimal fall. (refer to section 6.4 for further comment).

The Trimdek cladding should be subject to a performance guarantee. This would have to be checked with the current owners.

(F)

ALAN REAY CONSULTANTS LIMITED

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M.N.Z.I.E.
Registered Engineer
Structural Consultant

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File 3608.

FACSIMILIE TRANSMISSION

DATE: 2/02/90

TO: HOLMES CONSULTING GROUP

ATTENTION: GRANT WILKINSON

CITY: CHRISTCHURCH

RECEIVERS FAX NO: (03) 792-169

FROM: GEOFF BANKS.

MESSAGE:

RE: 249 MADRAS ST

Attached letter and sketch as discussed.

Regards, Geoff

PLEASE CONTACT IMMEDIATELY IF 3 PAGES (INCLUDING THIS) ARE NOT RECEIVED.

BUI.MAD249.0130.12

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M.N.Z.I.E.
Registered Engineer
Structural Consultant

File 3608

2 February 1990

Mr Grant Wilkinson
Holmes Consulting Group
PO Box 701
CHRISTCHURCH

Dear Sir

RE: 249 MADRAS STREET

Further to our discussions by telephone this morning, we confirm that the scope of the possible non-compliance referred to in your report on the building is the connections between the walls on gridlines D and D/E, as shown on the attached sketch SK1 from levels 2 to 6 inclusive (Level 1 being the ground floor carpark).

The proposed remedial work, if required, would consist of a total of two ties per floor, tying the walls to the floor diaphragm.

The agreed maximum tie load is 300 kN per tie. We understand that this load would be reduced on lower floors in accordance with the "Parts and Portions" section of NZS 4203:1984.

Please contact this office today if your understanding of the situation is not as outlined above.

Yours faithfully

G.N. Banks

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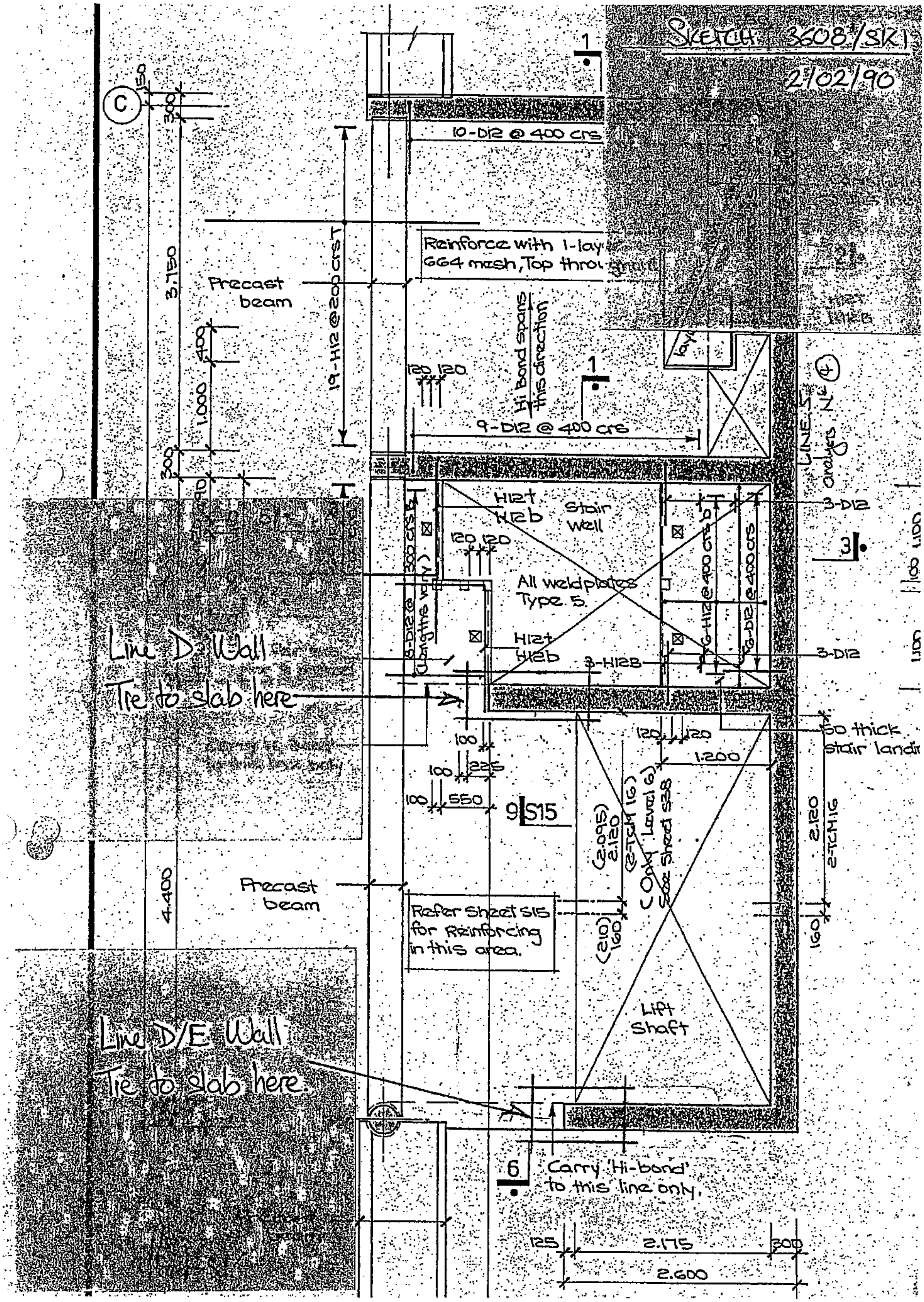

3808

14/02/90.

(R) John Hare, HUPT 9-00 am.

1 Agreed loads 300 L5
240 L4
184 L3, 2, 1.

2. Confirmed tie only system OK – possible pretension via nuts

3. Confirmed reduced connection L1 may be OK (could compensate at L2 if necessary). Will confirm so work proceeds

GB

CALCULATIONS

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CONSULTING ENGINEER
CHRISTCHURCH

249 MADRAS ST – DIAPHRAGM CHECK

PAGE 1
SECT
FILE 3608
DATE 29/01/90

- Building has 5 suspended floors and a lightweight roof.
- Check floor diaphragm connection to wall on line 5, line D and line E.

1. Geometry

refer sketch attached for layout.

2. Loads · refer 'SSS of previous calcs

2.1 Previous Calcs · checked line 1 & 5 walls only (level 5)
50% load to each
used factored static design shear
of 501 kN (storey), no overstrength.

2.2 Check Parts & Portions

storey weight = 6217 kN @ level 5.

$C_p^{max} = 0.3$ for multi-storey

$C_p = \alpha K_x Z R C_p$

$\alpha = \frac{h_{eq}}{h_n}$ $h_{eq} = \frac{\Sigma W_x h_x}{\Sigma W_x} = \frac{316105}{31766} = 9.95\,m.$

$h_n = 20.50$

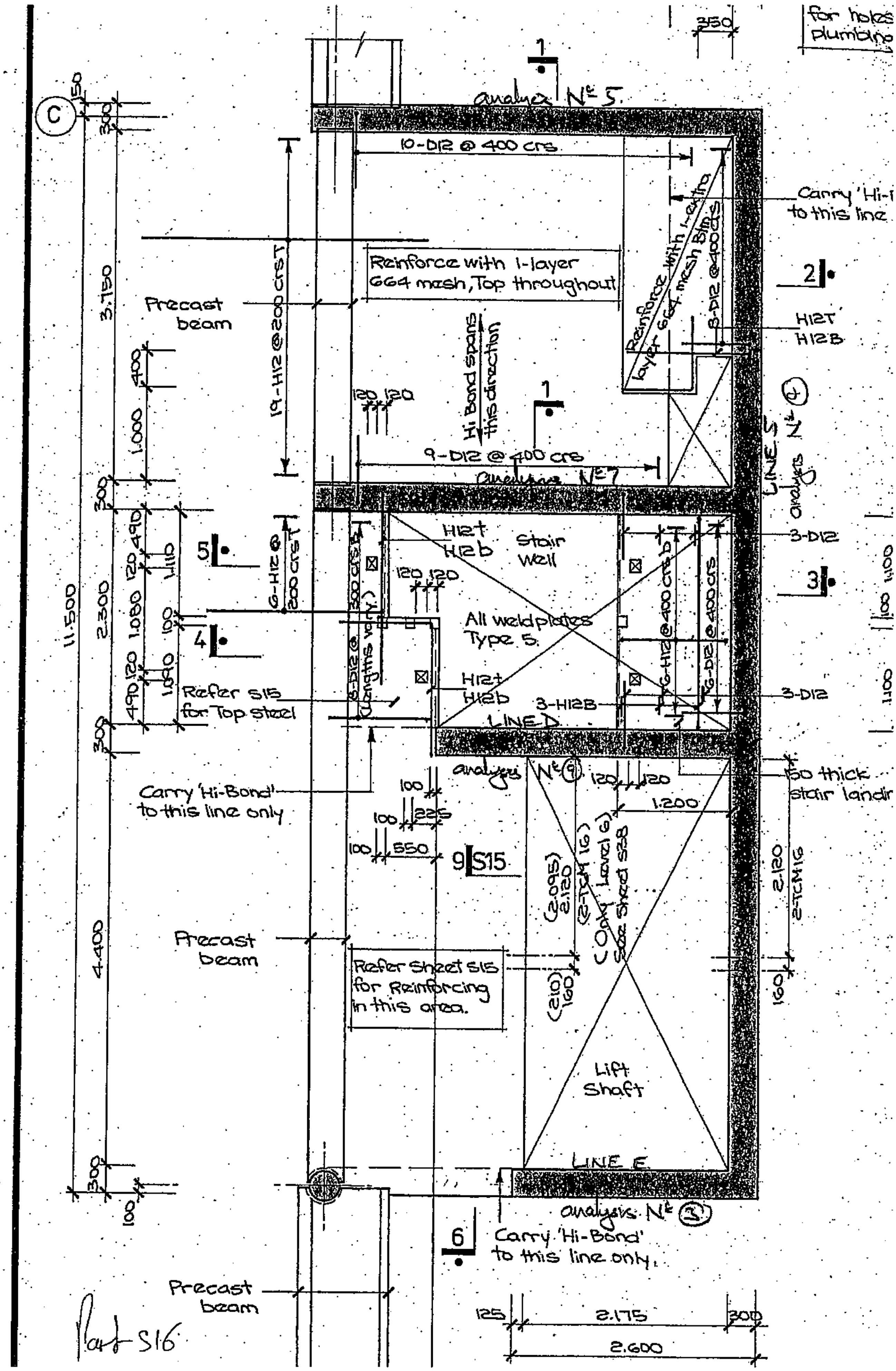
$\Rightarrow \alpha = \frac{9.95}{20.50} = 0.49$

top storey: $K_x = \frac{h_x}{h_{eq}} = \frac{16.20}{9.95} = 1.63$

$Z = 5/6$ zone B

$R = 1.0$

$C_p = 0.3$


for holes
plumbing
350
analysis No 5.
10-D12 @ 400 crs
Carry 'Hi-Bond'
to this line
Reinforce with 1-layer
664 mesh, Top throughout
Reinforce with 1-extra layer 664 mesh Btm
8-D12 @ 400 crs
Precast
beam
19-H12 @ 200 crs T
2
H12T
H12B
Hi Bond spans
this direction
120 120
1
9-D12 @ 400 crs
analysis No 7
LINE 5
analysis No 4
H12t
H12b
Stair
Well
3-D12
5
6-H12 @ 200 crs T
120 120
3
All weldplates
Type 5.
4
6-H12 @ 400 crs b
6-D12 @ 400 crs
Refer S15
for Top steel
H12t
H12b
3-H12B
3-D12
LINE D
Carry 'Hi-Bond'
to this line only
analysis No 9
120 120
1.200
50 thick
stair landing
100
225
100
550
9 S15
(2.095)
2.120
(2-TCM 16)
(Only Level 6)
See Sheet S38
2.120
2-TCM 16
Precast
beam
Refer Sheet S15
for Reinforcing
in this area.
(210)
160
160
Lift
Shaft
LINE E
analysis No 3
6
Carry 'Hi-Bond'
to this line only.
Precast
beam
125
2.175
300
2.600
C
150
300
3.750
400
1.000
300
490
120
1.080
120
490
100
100
1.010
1.010
11.500
2.300
300
4.400
300
100
100
100
100
Part S16

$\Rightarrow C_p = (0.49)(1.63)(5/6)(1.0)(0.3) = 0.20$

$\Rightarrow$ storey shear = (0.20) 6217
$= \boxed{1241 \text{ kN}}$

2.3 Storey Shear Distribution

2.3.1 Y-direction eq

· distribute according to static shear distⁿ at ground :

wall (4) (case 3) $\frac{1925}{3301}(1241) = 0.58 \times 1241$

$= \underline{\underline{724 \text{ kN}}}$.

2.3.2 X-direction eq

wall (3) (case 1) $\frac{613}{3298}(1241) = \underline{\underline{231 \text{ kN}}}$

wall (9) (case 1) $\frac{741}{3298}(1241) = \underline{\underline{279 \text{ kN}}}$

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3 Design Connections – Reinforcement

3.1 Wall ④

design V_u = 724 kN over 2.65 m length

shear friction to wall (roughened surface ≥5mm)

$$A_v = \frac{724 \times 10^3}{2.65(0.85 \times 1.4)275} = 835 \text{ mm}^2/\text{m}$$

have D12 @ 400 + 2 layers 664 mesh.

$$A_s^{equiv} = \frac{113}{0.4} + 2\left[186 \times \frac{485}{275}\right] = 939 \text{ mm}^2/\text{m} > 835 \Rightarrow ok$$

diaphragm

$$v_i = \frac{724 \times 10^3}{0.85 \times 150 \times 2650} = 2.14 \text{ MPa}$$

$$v_c = 0.2\sqrt{f'_c} = 1.00 \text{ MPa}$$

$$v_s = 1.14 \text{ MPa}$$

$$A_v = \frac{1.14 \times 150 \times 10^3}{485} = 353 \text{ m}^2/\text{m}$$

have 2 layers 664 mesh = 2×186 = 372 mm²/m ⇒ ok

wall line 5 connection ok

3.2 Wall ③

tension tie to slab : need $A_{st} = \frac{231 \times 10^3}{0.9 \times 380} = 675 \text{ mm}^2.$

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4. Design Connections - Bolted.

For walls 3 & 9, $T_u = 279$ kN max.

try fabricated angle:

bolts: (in shear) - use $f_{os} = 3.3$ for EQ

Chemset M20; $V_u = \frac{85.7}{3.3} = 26.0$ kN.

Truebolt T20; $V_u = \frac{85.3}{3.3} = 25.8$.

equiv.

$\Rightarrow$ N^o reqd $= \frac{279}{26.0} = 11$ — high!

try M24 Chemset, $V_u = \frac{127.6}{3.3} = 38.7$

$\Rightarrow$ N^o reqd $= \frac{279}{38.7} = 7.2$; say 7. (f_{os} 3.2)

$\boxed{\text{7-M24 Chemset Anchors}}$

steel area: $A_t = \frac{279}{.250} = 1116$ mm².

$\Rightarrow$ $\boxed{150 \times 12 \text{ m.s. flat}}$

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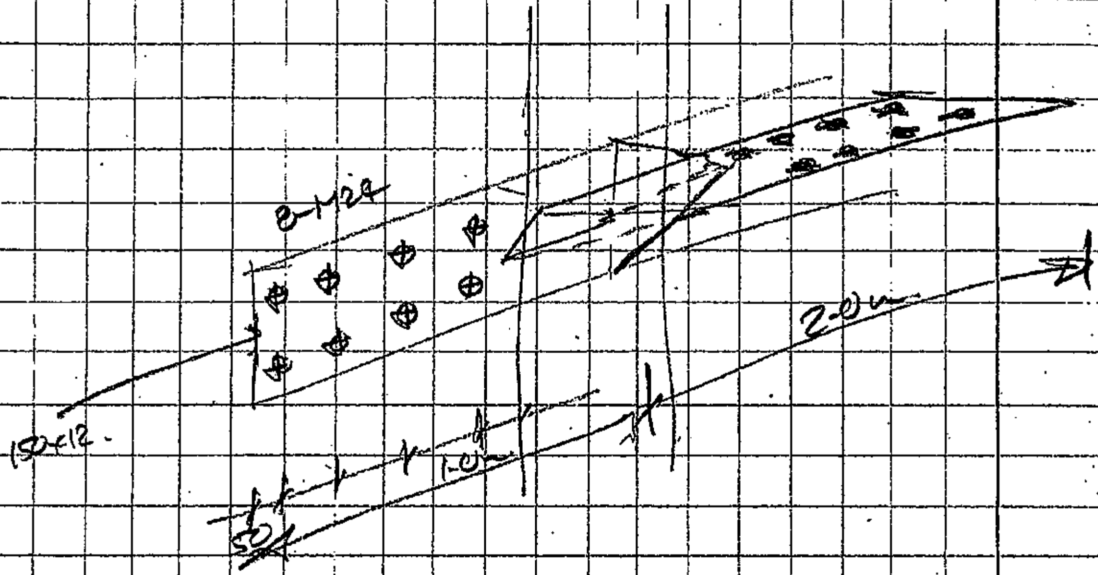

steel : 50 kg @ $2/kg = $100 steel

bolts + wax 15 @ $10 - $150 est.

labour. $340.

$600 /unit.

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5. Design Connection - Tie System.

- design tie load for walls ③ + ④ from 2.3.2 is 279 kN maximum, for Parts + Portions.
- agreed max design load with HWPT is 300 kN
  ⇒ use this as a maximum at level 5 (of analysers model) and reduce for lower levels.

5.1 Loads

		Udes.
at level 5: $\alpha K_x = (0.49)(1.63) = 0.80$		300 kN
at level 4: $\alpha K_x = (0.49)\left(\frac{12.96}{9.95}\right) = 0.64$	⇒	240 kN.
at level 3: $\alpha K_x = (0.49)(1.00) = 0.49$	⇒	184 kN.
at level 2: " " "		184 kN.
at level 1: " " "		184 kN.

5.2 Compression Transfer

- check whether slab/wall junction can handle compression transfer without reinforcement:

$$f_c^{max} = \frac{300 \times 10^3}{300 \times 200} = 5.0 \text{ MPa}$$

conservative

allowable $F_c = (0.85 \times 0.70)250 = 14.9$ MPa

⇒ compression easily ok.

5.3 Tension Tie

5.3.1 Wall Nº 9.

- use angle bolted to wall & floor as previous detail
- 2-H12 ties located in the wall end at level 2.
- capacity of these 2 bars is:

$$T_u = 0.9(.113 \times 380)2 = 77 \text{ kN}.$$

- check if level 2 & 1 can be transferred to other walls (see section 6), ⇒ design bracket for the following loads:

				M20	M24 Chemset
level 5:	300 − 77	=	223 kN.	9	6
level 4:	240 − 77	=	163 kN	6	5
level 3:	184 − 77	=	107 kN.	4.	3

65 ~~8mm~~

- try using M24 Chemset Anchors to wall.
  Ultimate shear load = 127.6 kN.
  - load case 0.8 E
  - use f.o.s of 4.0 for working loads ⇒ for loads above, use 0.8 × 4.0 = 3.2.

  ie $V_{M24} = \frac{127.6}{3.2} = 40$ kN.

⇒ Nº bolts shown above

- recommended spacing = 160 mm.

⇒ detail bracket as shown over.

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PAGE 8A.
SECT
FILE 3608
DATE 10/10/91.

• use M20 anchors to floor

$V_{M20} = \frac{85.7}{3.2} \approx 27$ kN. (see 7A for N's)

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PAGE 9A
SECT
FILE 3608.
DATE 10/10/91.

5.3.2 Wall No 3

as wall 9, except only 1 H12bar per level located.

⇒ design loads as follows:

					H20	M24
level 5:	300 - 38	=	262 kN		10	7
level 4:	240 - 38	=	202 kN		8	5
level 3:	184 - 38	=	146 kN		6	4

⇒ bolt numbers as noted:
(Chemset)

6 Shear Transfer, Levels 1 and 2

- check whether loads in diaphragms at these levels can be transferred to walls 5 and 7

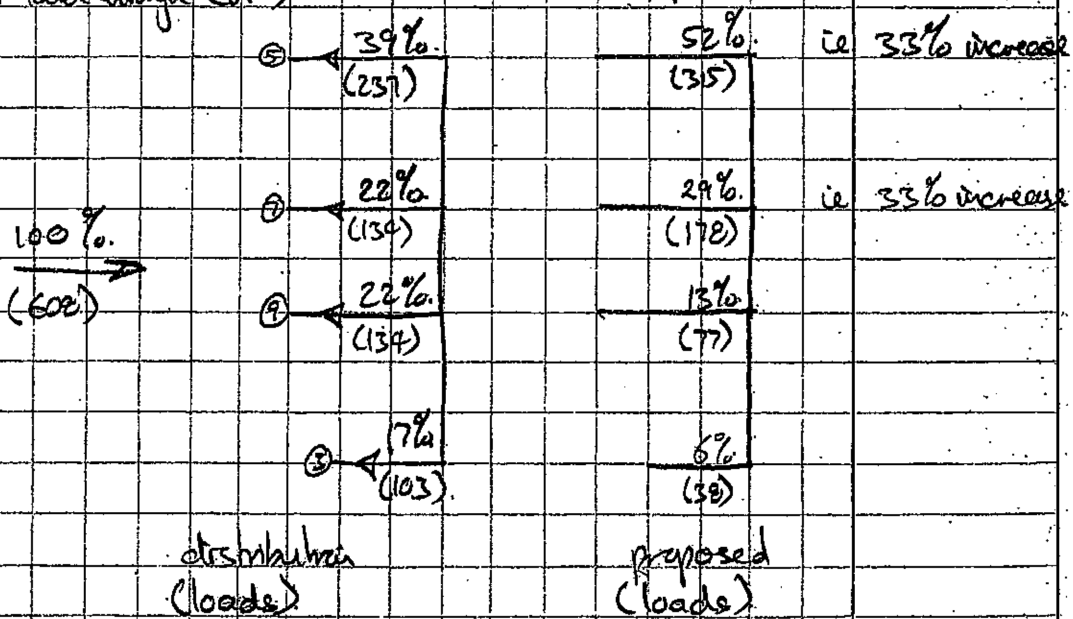
- design shears for diaphragm design are as follows:

max storey shear — reduction for height

level 2 : 1241 (0.49) = 608 kN

level 1 : 1241 (0.49) = 608 kN.

- from page S12 of calcs, previous static dist^n of storey shears was as follows: (x direction) (at load through CoM)



- strength of connection to 3 is a min of 38 kN.
  " " 9 77 kN.

⇒ proposed revised distribution above.

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Christchurch

Check the effect of this increase on wall flexural and Shear Design :

Wall N^o 5 : ref. p347A of calcs.

code achieve : applied shear — incr 33%

at level 2 : V_{code} = 870−762 = 108 kN. — 144 kN.

" " 1 : = 675−870 = −195 kN — −259 kN.

⇒ max V_{code} = 762 + 144 = 906 kN at level 1→2.

Flexure

and max M_{base} = 4267+36(6.9) − 64(3.7) = 4279 kNm

ie 0.2% increase !

negligible

⇒ no increase required in flexural capacity

shear : ref p347B.

	C	T
at level 2 : new v_i =	4.34	2.54
v_c	2.07	0.55
v_s	2.27	1.99
A_v	1791 mm²/m.	

· have H16 @ 250 e.f. = 1608 mm²/m. 90%

· but note wall length 38% longer than assumed in calc ⇒ ok

shear ok

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Christchurch

PAGE 14A
SECT.
FILE 3608
DATE 10/10/91

Wall No 7 : • refer to p348 of calcs

code actions:
• info not available on actions above ground floor.

however :

flexure : max. possible moment increase is (178-134)(6.9+3.7) = 466 kNm. on design Mode 3977 kNm, ie 12% incr.

• but detail for ductility => OK.

(& likely bar strength margin = 430/380 = 1.13) => OK.

shear : governed by overstrength
- details as wall 7, but design shear less => OK.

$\boxed{\text{flexure + shear OK}}$

BUI.MAD249.0130.28

ALAN REAY CONSULTANTS LIMITED

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Fax No. (03) 793-981

FAXED

FACSIMILE TRANSMISSION

File 3602.

DATE: 10/10/91

TO: CBD CONSTRUCTION

ATTENTION: JIM FRY

CITY: CHCH

RECEIVERS FAX NO: 384 2817

FROM: GEOFF BANKS

MESSAGE: 249 MADRAS ST

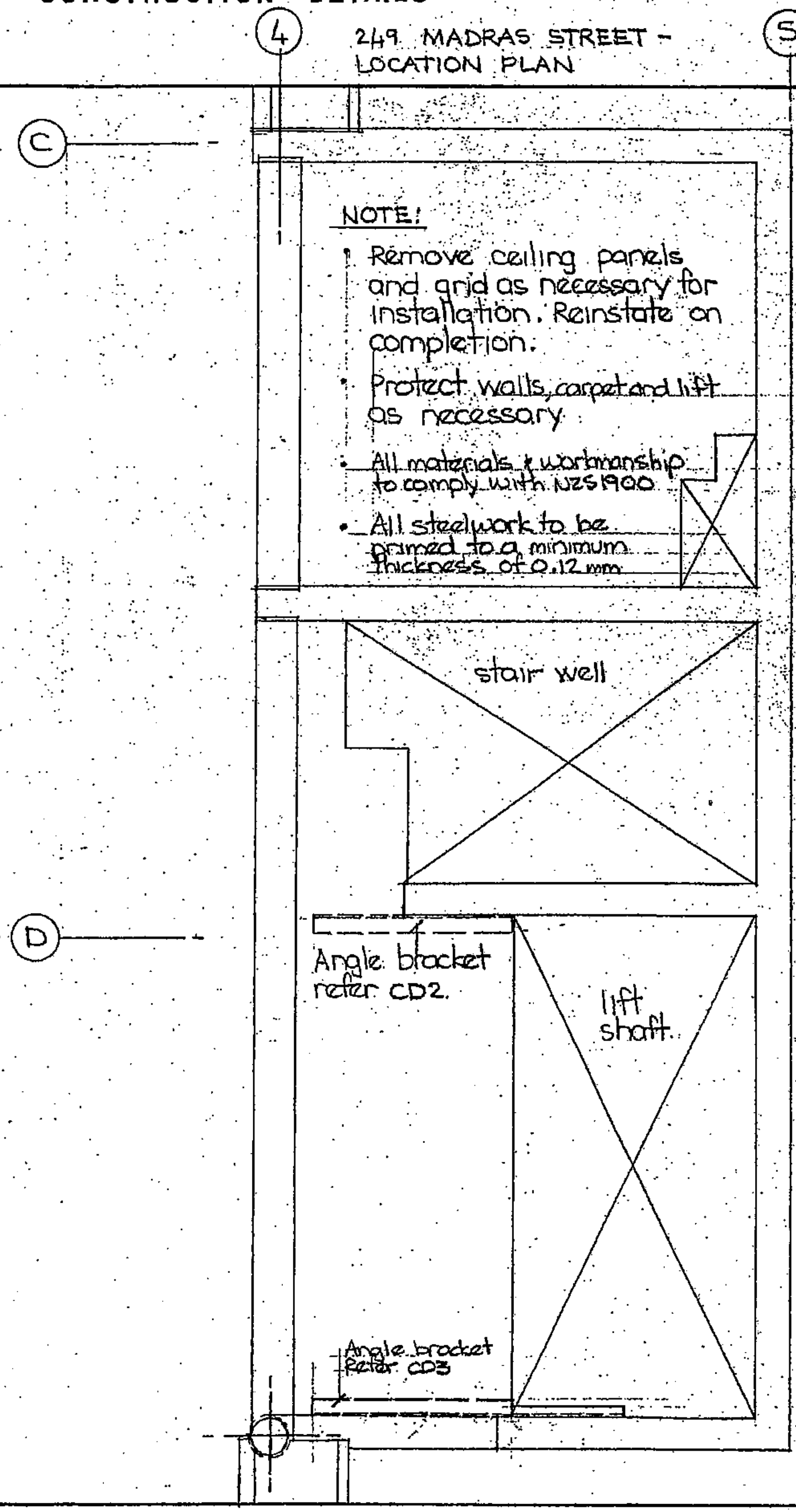
- Attached are our updated details as discussed.
- Contact Barry O'Neil at Otis re access for lift.

PLEASE CONTACT IMMEDIATELY IF 4 PAGES (INCLUDING THIS) ARE NOT RECEIVED

CONSTRUCTION DETAILS

249 MADRAS STREET –
LOCATION PLAN

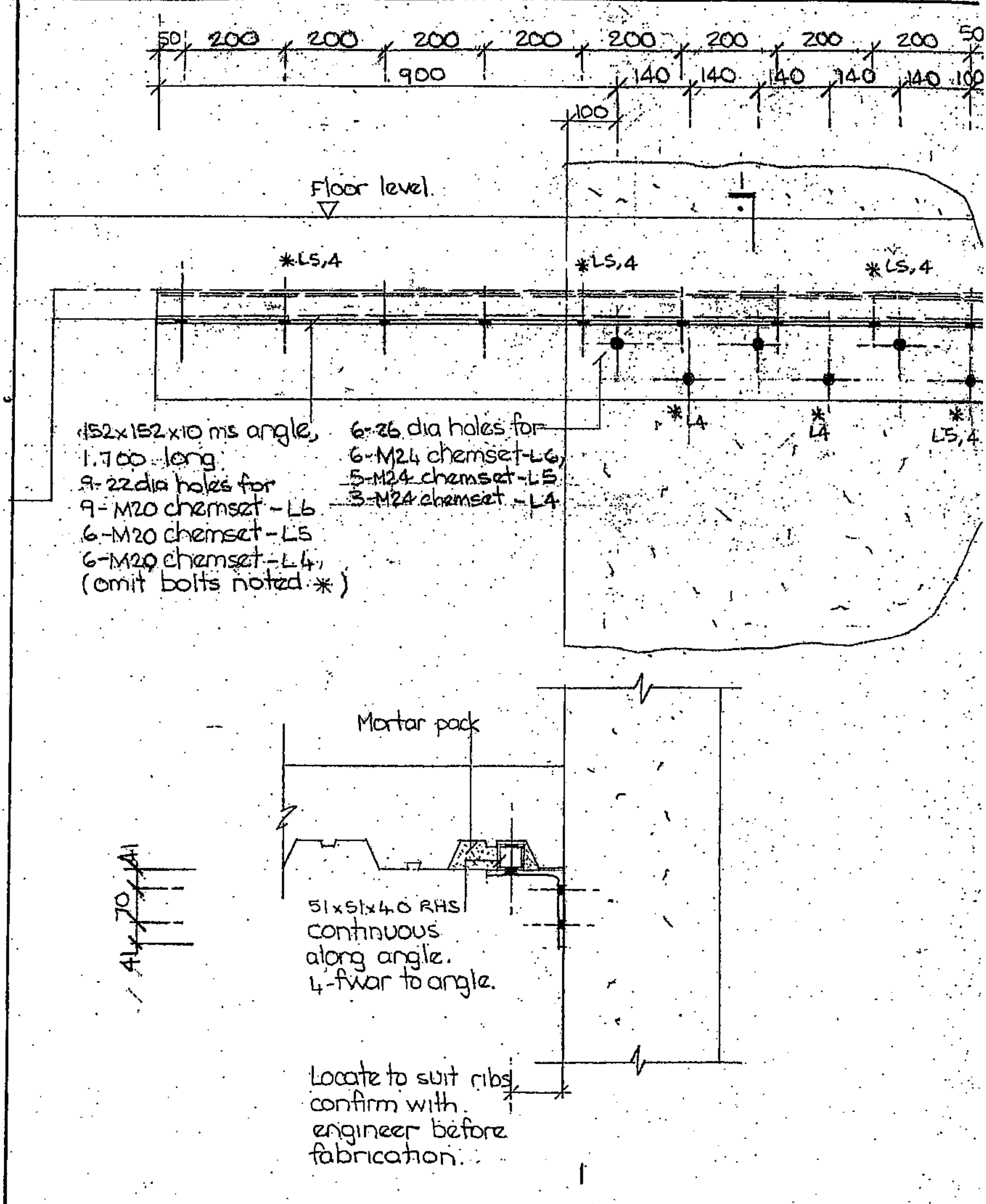
PAGE	CD1
SECT	WALL TIES
FILE	3608
DATE	FEB 1990



CONSTRUCTION DETAILS

WALL TIE TO SLAB. GRID LINES D, 4/5
LEVELS 4-6

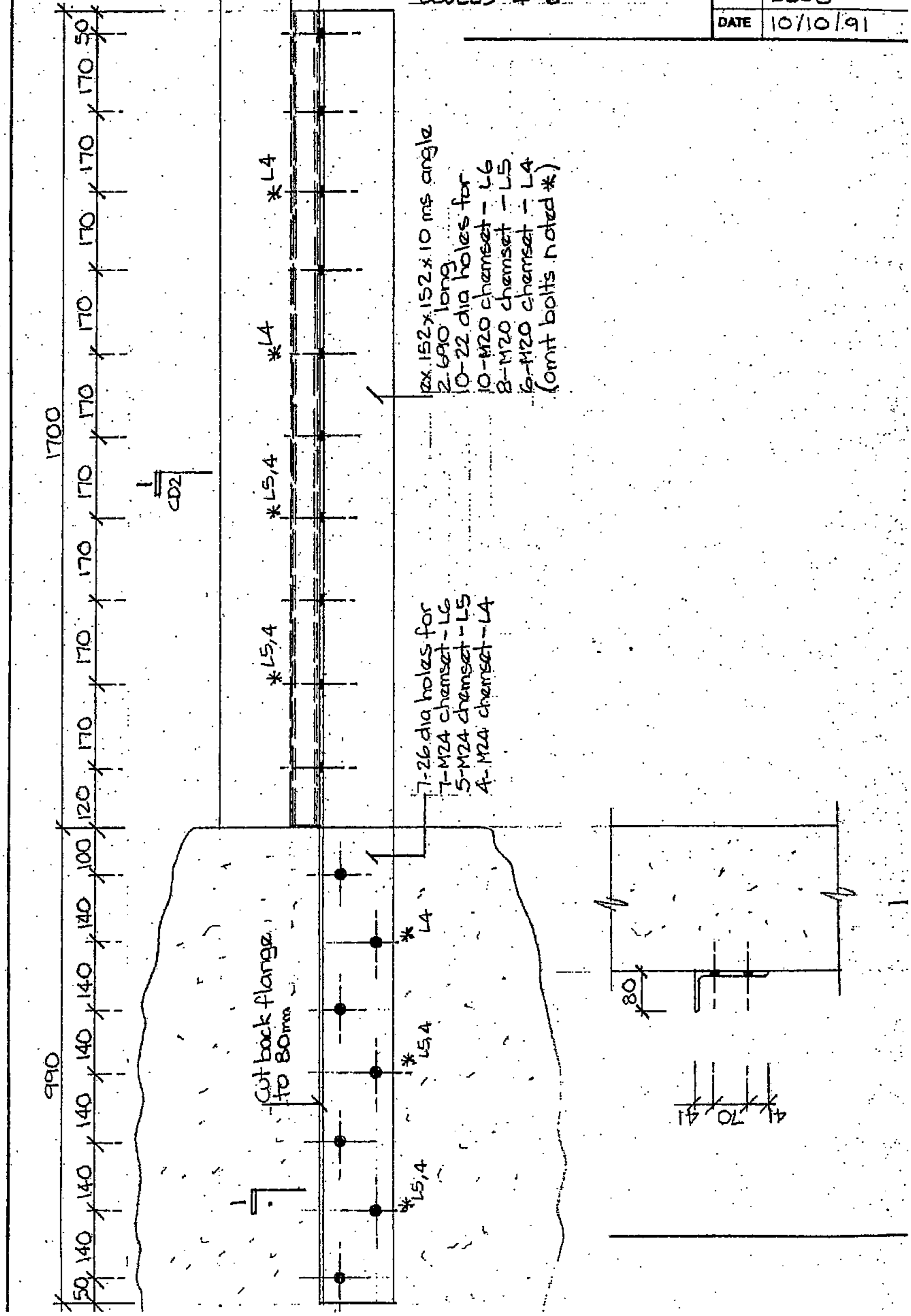
PAGE	CD 2
SECT	WALL TIES
FILE	3608
DATE	FEB 1990



CONSTRUCTION DETAILS

WALL TIE TO SLAB
GRID LINES D/E, 4/5.
LEVELS 4-6

PAGE	CD3
SECT	WALL TIES
FILE	3608
DATE	10/10/91



ALAN REAY CONSULTANTS LIMITED

147 KILMORE ST
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Telephone (03) 660-434
Fax No. (03) 793-981

FILE 3608

11 October 1991

Mr M. Rogers
Ministry of Transport
Marine and Industrial Section
Private Bag
CHRISTCHURCH

Dear Sir

RE: LIFT SHAFT - 249 MADRAS STREET

As discussed with you yesterday, we confirm that we wish to install a structural steel angle in the lift shaft at levels 6, 5, and 4, as shown on the attached sketches 3608/CD1, 2, 3. The angle is a structural tie only, and is not supporting any additional services.

We understand that this detail is acceptable to you, and that the work may proceed.

Please contact the writer if you have any further queries.

Yours faithfully

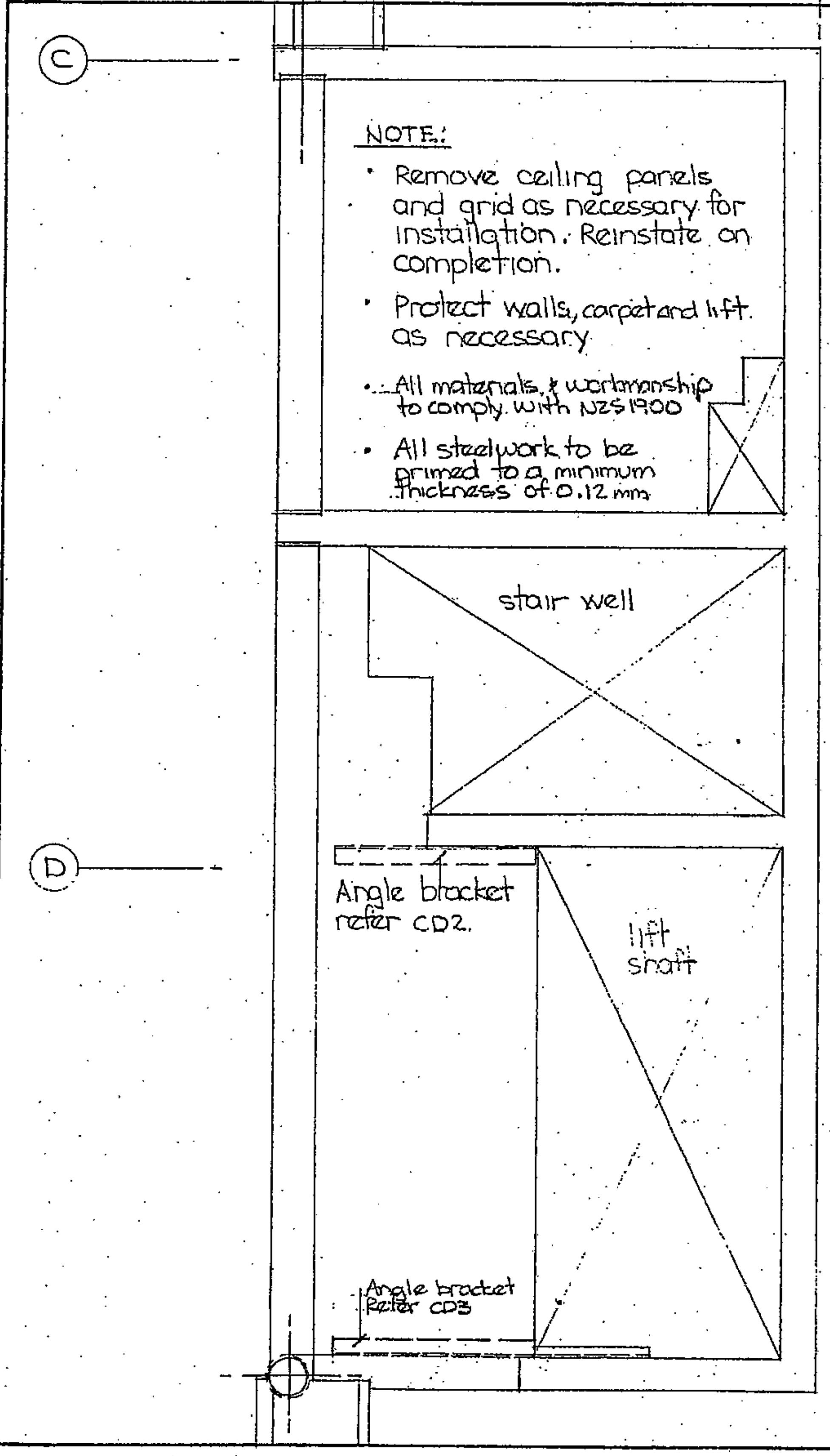
G.N. Banks

Encl:

CONSTRUCTION DETAILS

249 MADRAS STREET - LOCATION PLAN

PAGE	CD1
SECT	WALL TIES
FILE	3608
DATE	FEB 1990

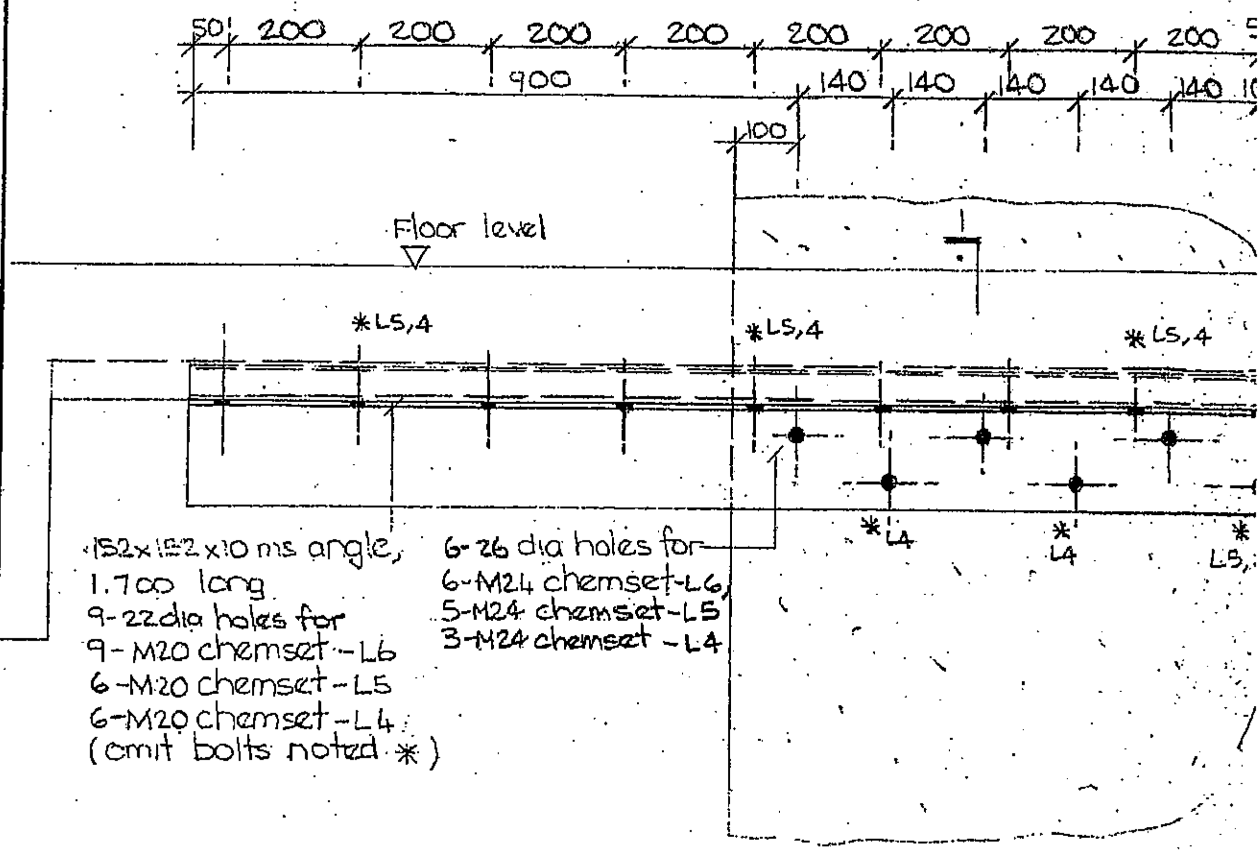


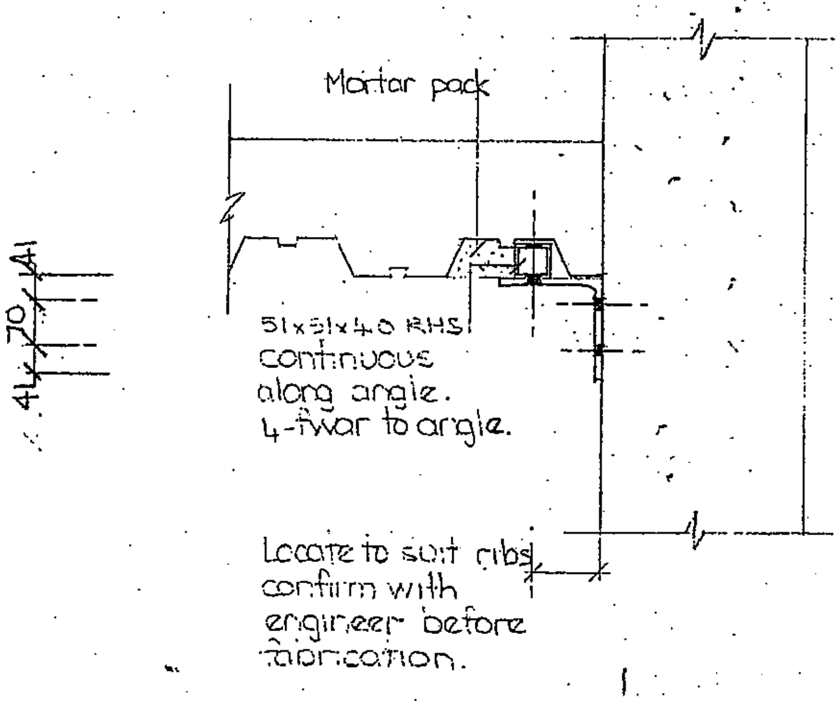
ALAN M. REAY Consulting Engineers

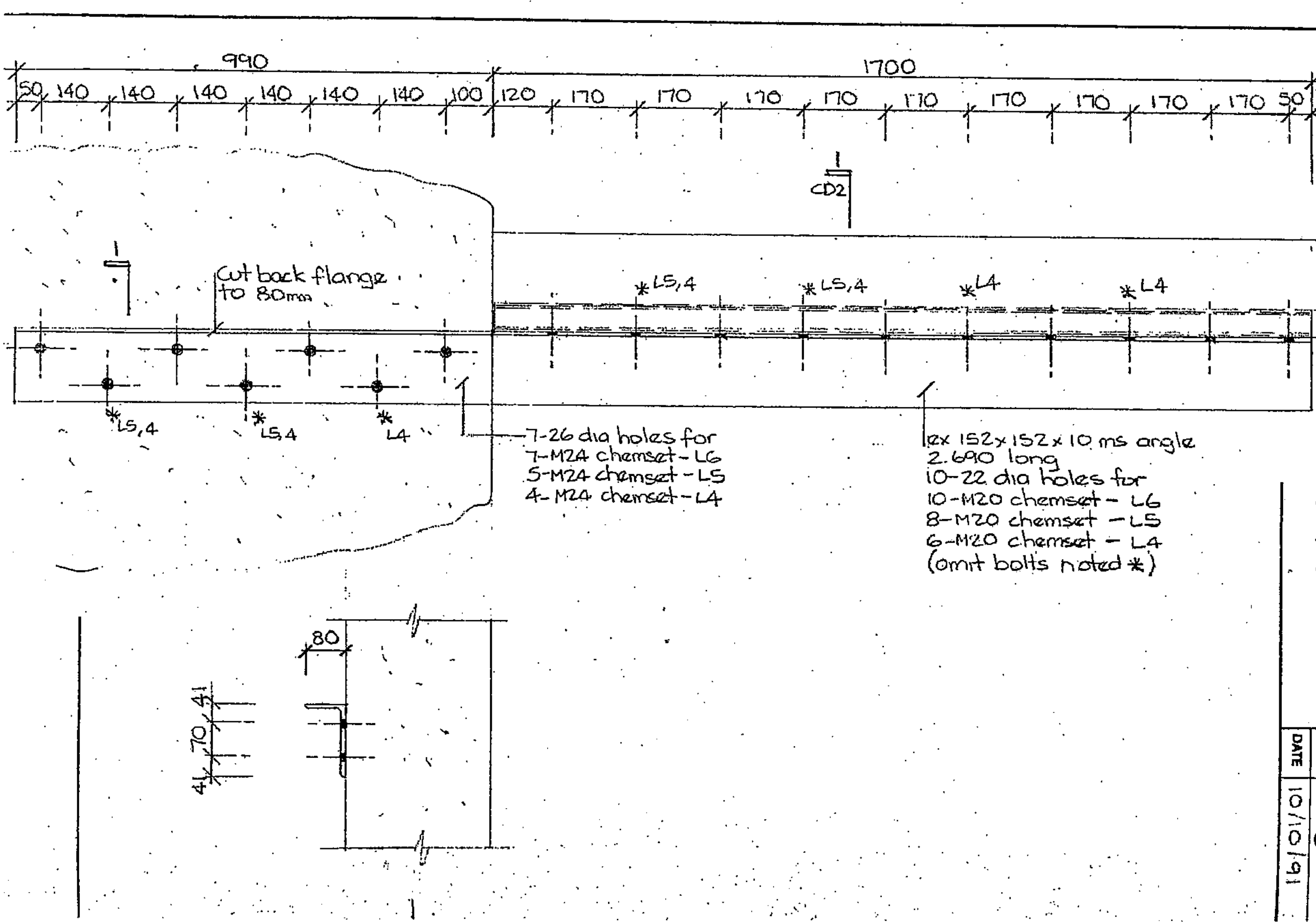
CONSTRUCTION DETAILS

WALL TIE TO SLAB. GRID LINES D, 4/5
LEVELS 4-6

PAGE	CD 2
SECT	WALLTIES
FILE	3608
DATE	FEB 1990







CONSTRUCTION DETAILS

WALL TIE TO SLAB
GRID LINES D/E, 4/5,
LEVELS 4-6

PAGE	CD3
SECT	WALL TIES
FILE	3608
DATE	10/10/91

MARINE & INDUSTRIAL

M&I

SAFETY
INSPECTION
SERVICES

Ngā Ratonga Ārai
Mate Ahumahi,
Taimoana

FILE	3608
DATE	21.10.91
AMR	

Our Ref : 41/3/1

Your Ref : 3608

18 October 1991

Alan Reay Consultants Ltd
P.O. Box 25-028
Victoria Street
Christchurch

Attention:- G. N. Banks

Dear Sir

LIFT SHAFT - 249 MADRAS STREET

Thank you for your letter of 11 October concerning the above lift shaft.

I confirm that the details as shown on sketches 3608/CD1, 2, and 3 are acceptable.

Yours faithfully

RM Rodgers

R. M. Rodgers
For Southern Operations Manager

A Service of the
Maritime Transport Division

Ministry of Transport
Te Manatū Waka

151-153 KILMORE ST
PRIVATE BAG
CHRISTCHURCH
NEW ZEALAND
TELEPHONE (03) 635-635
FACSIMILE (03) 665-748

C.B.D.
CONSTRUCTION
LIMITED

P.O. Box 10-318, Christchurch. Telephone (03) 842-455, Fax (03) 842-817

FILE	3608
DATE	15.10.91
AMR	

FAX FROM: GRANT BLACKMORE

FOR: JEFF BANKS

OF: ALAN REAY CONSULTANTS LIMITED

FAX NUMBER: 793 - 981

DESTINATION: CHRISTCHURCH

NO. OF PAGES (INCLUDING THIS HEADER): 2

PLEASE PHONE 842-455 IF THE PAGES ARE NOT WELL RECEIVED.

MESSAGE:

CBD
CONSTRUCTION
LIMITED

P.O. Box 10-318, Christchurch. Telephone (03) 384-2455, Fax (03) 384-2817

15 October 1991

Alan Reay Consultants Limited
P O Box 25-028
CHRISTCHURCH

Dear Sir

RE: 249 MADRAS STREET

Our quotation to supply and fix angle brackets as per details including remedial to floors broken out for investigation, $4,633.50 (FOUR THOUSAND SIX HUNDRED AND THIRTY-THREE DOLLARS AND FIFTY CENTS ONLY) Plus G.S.T.

Yours faithfully

G.M Blackmore
QUANTITY SURVEYOR