HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS

Fax Number 654 392 Job No W 8165

Job Name 249 Madras st Structural report

To Robin Schulz of Schulz Knight

From John Hare

Time 10-40 Date 31/1/90

Re:

Please see over a draft copy of our report, for your information & comment

Regards
John Hare.

Holmes Consulting Group Limited,
61 Cambridge Terrace,
P.O. Box 701,
Christchurch, New Zealand.
Telephone: (03) 663-366.
Facsimile: (03) 792-169.

Offices in Christchurch, Wellington, New Plymouth, Auckland.

W8165

ALAN REAY CONSULTANTS LIMITED

147 KILMORE STREET
BOX 25-028, VICTORIA ST
CHRISTCHURCH 1
Telephone: 660-434
Fax No: (03) 793-981

ALAN M. REAY
B.E.(Hons.), Ph.D.
M.N.Z.I.E.
Registered Engineer
Structural Consultant

File 3608

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- 7 FEB 1990
HOLMES CONSULTING GROUP LIMITED CHRISTCHURCH

RAP
JMH
RGW
HJH

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.

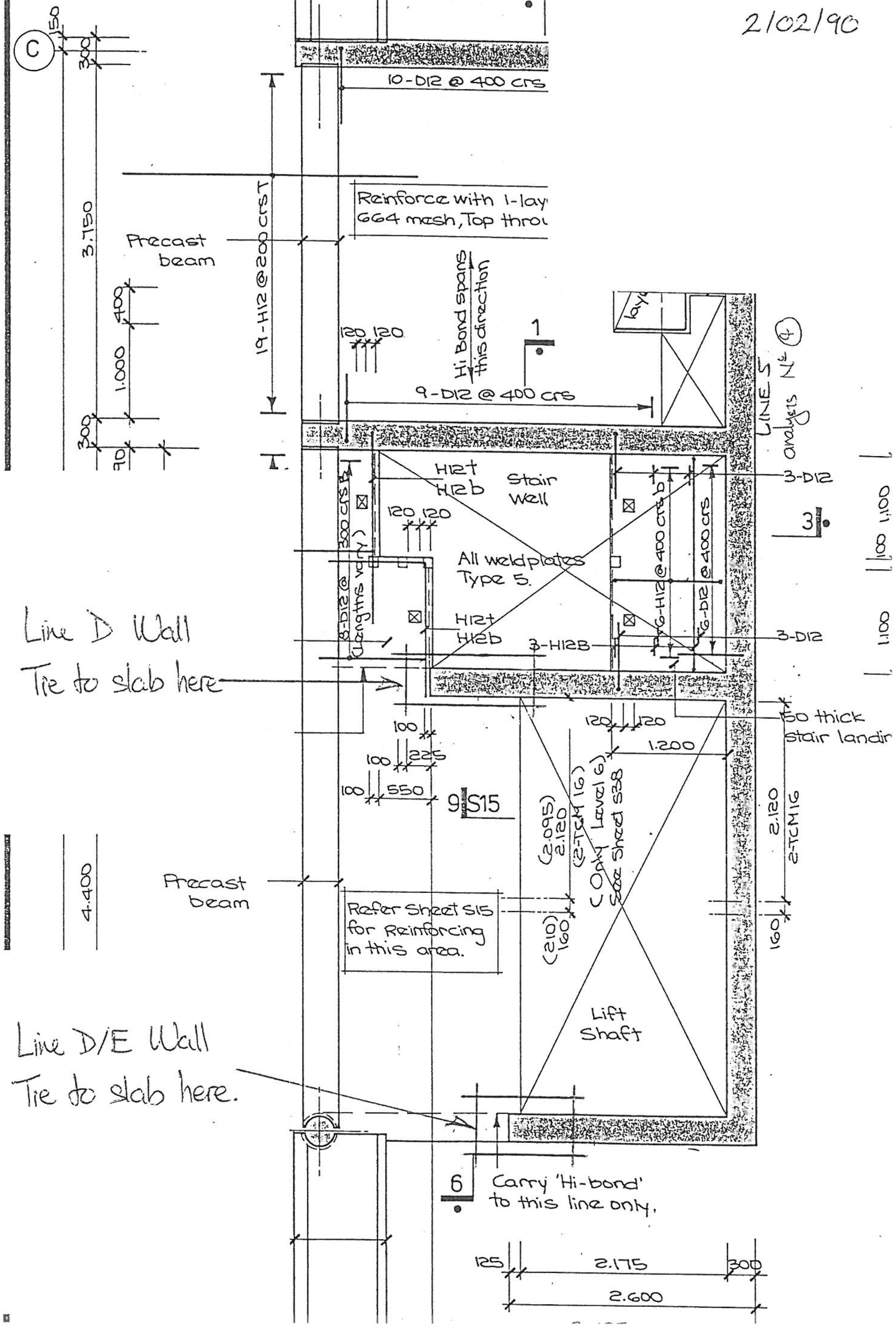
Please contact this office today if your understanding of the situation is not as outlined above.

Yours faithfully

G.N. Banks

SKETCH 3608/SK1

2/02/90



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Structural Consultant

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G

→ MJH

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

ALAN REAY CONSULTANTS LIMITED

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2 February 1990

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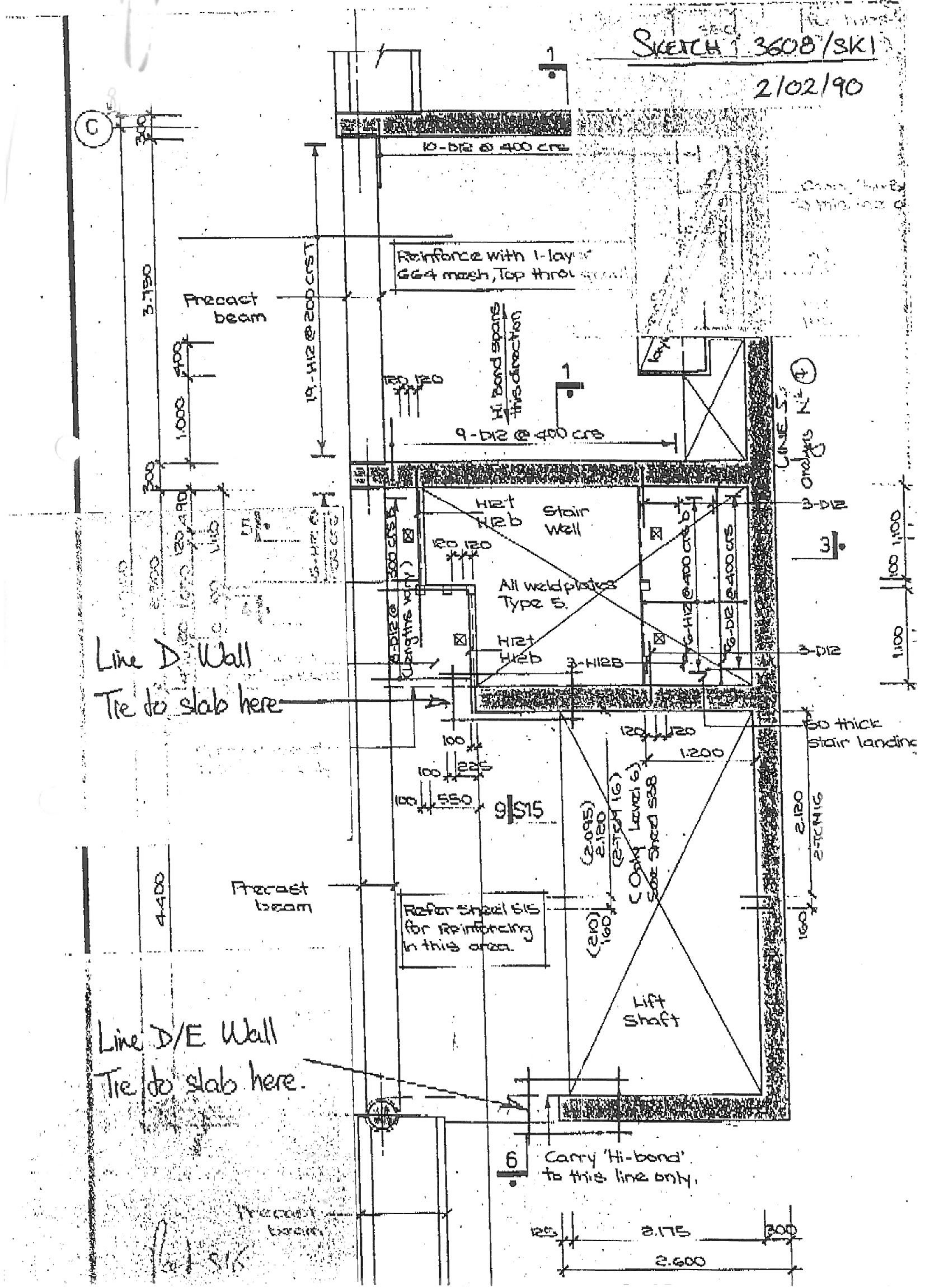

SKETCH 1 3608/SK1
2/02/90
10-D12 @ 400 crs
Reinforce with 1-layer
664 mesh, Top throu
Precast beam
19-H12 @ 200 crs T
Hi Bond spans this direction
9-D12 @ 400 crs
3.790
1.000
300
400
Stair Wall
H12t
H12b
120 120
All weld plates Type 5.
3-D12
3-H12B
6-H12 @ 400 crs b
6-D12 @ 400 crs
1,100
1,100
Line D Wall
Tie to slab here
150 thick stair landing
1.200
100
225
550
9 S15
2.120
2-TCM 16
(2.095)
C Only Level 6
See Sheet S38
4.400
Precast beam
Refer Sheet S15 for Reinforcing in this area.
(210)
160
Lift Shaft
Line D/E Wall
Tie to slab here.
Carry 'Hi-bond' to this line only.
6
125
2.175
300
2.600
Precast beam

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,
P.O. Box 701,
Christchurch, New Zealand.
Telephone: (03) 663-366.
Facsimile: (03) 792-169.

Offices in Christchurch, Wellington, New Plymouth, Auckland.

CONTENTS

W8165REP **JANUARY 1990**

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.

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MEMO

JOB NAME 249 Madras St.

JOB No W8165

DATE 01/02/90

To: Warren & Mahoney

Attn: Kerry Mason

Kerry

Martin Charles advises that the cost ~~do~~ to carry out the remedial structural works (as per attached sheets) will be approx $14,000 + GST.

Do you need anything else from us on this job?

Regards

Grant Wilkinson.

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JOB No W8165
DATE 01/02/90

Remedial Works

Steel angle required
2 per level at 5 levels.

- 4 levels have suspended tile ceilings in grid.
- 1 level has plasterboard + paint ceiling

- assume all levels have carpet.

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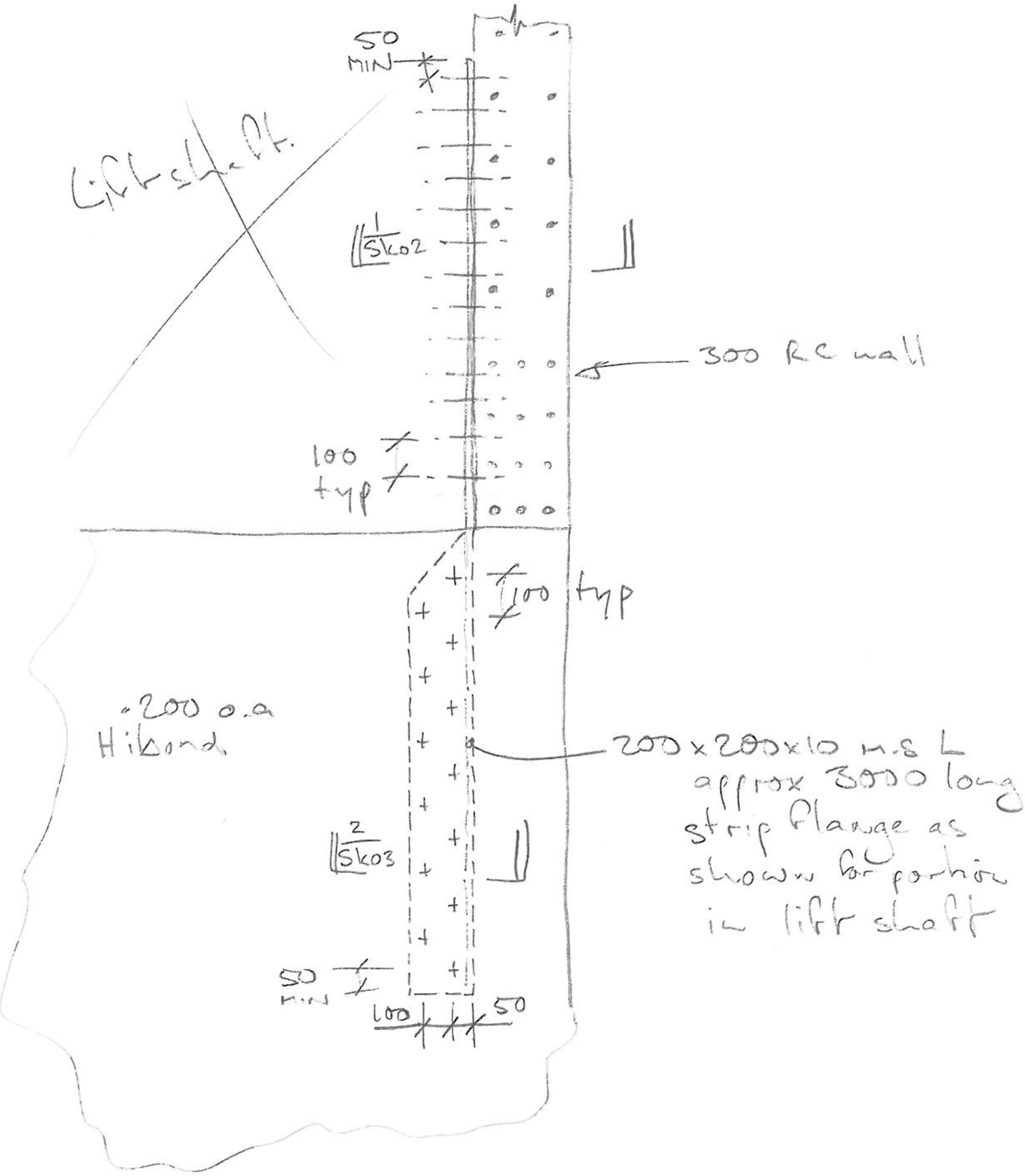
JOB NAME

PAGE Sk-01

JOB No

CALCS BY

DATE 31/1/90.



Plan 1:50 approx

- Require similar detail but handed at other side of lift shaft.
- Detail typical to levels 1 to 5.
- Reinstate all finishes when work completed eg ceilings, carpet.

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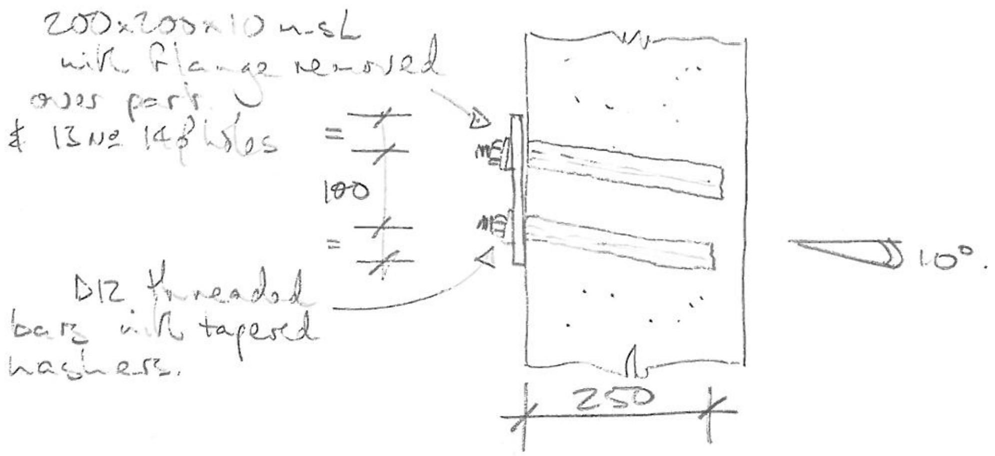
JOB No — CALCS BY — DATE



- Mark out & drill holes before cutting or drilling steel. Locate steel in wall prior to drilling holes.

- Holes in wall to be φ drilled using percussive rotary drill. Epoxy bars using Expocrete 'S' or equivalent.

1
Sk-01

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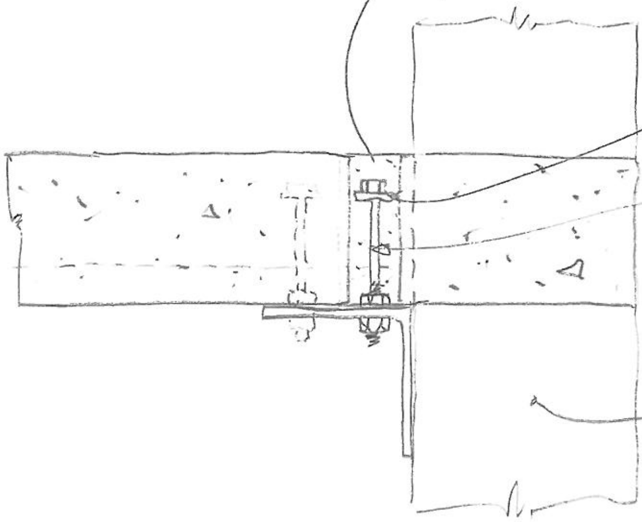

- Prior to drilling holes locate H12-200 bars top steel in slab. Confirm detail with engineer if any clash occurs.
- Mark out and drill holes through slab prior to cutting steel.
- After placing bolts temporarily fix angle to underside of slab, and seal around gaps between 'Hibond' ribs.
- Grout gap using SIKA 212 or similar

2 / Sk01

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Remedial work to N/S walls at lift shaft.

$T_i\phi / C_i\phi \approx 300$ kN either way.

⇒ Wall 300 thk ∴ practically only deep enough to develope D12.

∴ with $A_{vf} \geq \frac{300}{0.7 \times 300} = 1429$ mm²

∴ reqd $N \geq 1429/113 = 12.6$

⇒ Similar area of steel reqd under slab.

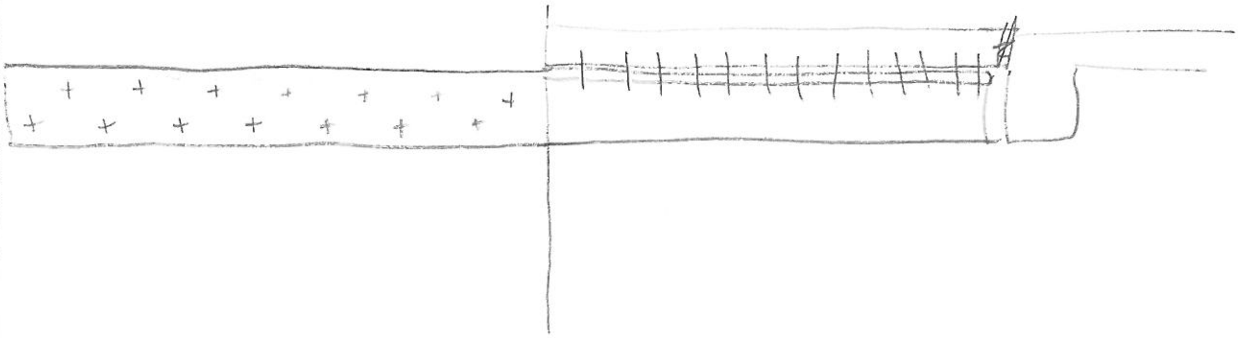
with angle tie less area of holes & 1 leg

~ $A_t \geq 300/250 = 1200$ mm².

∴ @ 10 thick, 120 well

∴ less 2 rows holes

⇒ 200 × 200 × 10 ms L



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JOB NAME W 81 65

JOB No

DATE 29/1/90.

Bryan Block 2-15

- Check legal definition on S. side. easements may or may not have been created.

dubious on quality of construction of Fire escape.
- some welding v. bad in 10 yrs. time will have construction faults.

- "Terrible elevation" - check vehicle clearance.

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MEMO

JOB NAME 249 Madras st.

JOB No

DATE 26/1/90

Documents @ Alan Reay.

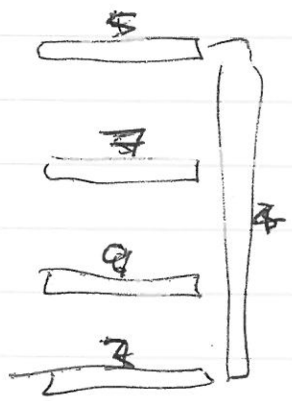
Gravity typically LL = 2.5 kPa. - possibly capacity up to 3.4 kPa. - could check.

Cd = .1 as calc.
- Static analysis

Static ΣV = 3300 kN OTM = 47254 kNm.

$T_x = T_y = 1.06$ sec ∴ ×.712 ∴ better.

Dynamic analysis performed.



1 ▯

2 ▯

1,2	P_D = 1493	P_L = 182
3	461	35
4	2100	
5	2073	234
7	1898	351
9	541	

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Wall 3 OTM_{code} 1646 kNm. V_{code} = 349
cf calc ~~P_{90} = 405~~
P_{90} = 496.

Wall 4 M_{code} = 8662 kNm ?

Wall 5 - max $M_i \phi$ = 16278
cf calc 15873.

Diaphragm calcs performed p S 56,57

- used bldg Cd modified for ~~x~~ linear.
T x .6 for wall @ edge = 300 kN max

- @ line 1 & 4 only
- no drag bars for other walls

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Foundations

→ Pads on silt.

Soils Investigation by S&F 18 June 86.

rec shallow ~~pad~~ footings or piles.
~ reduced pressure NE cnr for rubbish.

bearing stresses range 100 – 200 kPa @ 2500 sq
allowable.

- used 200 req'd stresses < 50–125
to limit settlement.

- Pads appear to be @ ≈ 180 kPa for 4^2

Designed beams to S~1 = 1.6

↳ not line 1 – used overstrength

~ seems ok

- Look @

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Roof steel

178 x 76 RSJ @ 7.00 span
@ 7300 crs!

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APPROX SEISMIC ANALYSIS.

Floors all typical

⇒ 200 Hibond on precast beams 400×550 o.a
with Core shear walls ↳ or 960×550 o.a
↳ on 400 ϕ columns.

⇒ A floor = 30.9 × 23.5 = 726 m² approx.

ℓ_{beams} = 74.5 m 400×550(350)

+ 60.5 m 960×550(350)

ℓ_{walls} = 26m @ 300
+ 4m @ 400. ← Coupled.

columns = 20 + ltwgt wall ℓ = 93 m.

∴ Wt = 726 × 4.5 Floor
+ 74.5 × .35 × .4 × 25 400×550 beams
+ 60.5 × .96 × .35 × 25 960×550 "
+ 26 × .3 × 3.24 × 25 300 walls
+ 4 × .4 × 3.24 × 25 400 walls
+ 20 × π × .4²/4 × 25 × 2.69 columns
+ .3 × 93 × 3.24 cladding

P_D = 5057 kN

h = 2.6 A Level 7 = 7 × 4.5 (150 Hibond) + 300 walls P_D = 600 kN

h = 2.4 A Level 8 = 11.5 × 4.5 + some walls + lightweight over P_D = 700 kN.

take live load 2.5 kPa typical ∴ $P_L/3$ = 605 kN

+ 5.0 kPa @ lvl 7, 8 ∴ $P_L/3$ = 53 kN 7

= 86 kN 8.

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∴ have ΣWt = 5(5057 + 605)
+ (600 + 53) + (700 + 86)
= 29750 kN

@ Cd: CRSM = .125 × 1.0 × 1.0 × 0.8
= 0.1 or .8 for coupled wall
∴ V base = 2975 kN.

∴ @ worst, .6 V/wall for E/W action
.5V N/S action

Dist

Lvl		W	h	Wh	F	Fh
	8	786	21.20	16663	163	3456
	7	653	18.80	12276	120	2256
	6	5662	16.20	91724	897	14531
	5	"	12.96	73380	718	9305
	4	"	9.72	55035	538	5229
	3	"	6.48	36690	359	2326
	2	"	3.24	18345	179	580
		29750		304113	2974	37683.

h CofG = 10.22 m.

→ Look @ diaphragm. @ Lvl 6
$S_p = 1.0$ ∴ $C_{p\,max} = 0.3$
∴ $C_p = \underbrace{\alpha\, K_{x}}\, Z\, R\, C_{p\,max}$
= 0.76 × 5/6 × 1.0 × 0.3
= .19.

∴ @ .6 Vstorey to wall - V = .19 x .6 x 5662
= 645 kN.

⇒ this load must go via small section of slab = 2600 long x 200 thick

⇒ v_i = 645/.2 x 2.6 = 1.24 MPa.

~~⇒ if~~ v_c = 1.18 MPa - o.k.

⇒ assuming overstrength not met

⇒ $v_i = \frac{2 \times 645}{.85 \times .2 \times 2.6}$

= 2.92 MPa

∴ $v_c/2$ = .59 MPa

∴ $A_v/s \geq \boxed{1554 \text{ mm}^2/\text{m}}$.

ie D12s - 70 crs.

not 400 !

OR, ⇒ say overstrength = 2.0 max

⇒ $v_i = \frac{2 \times 645}{.2 \times 2.6}$ = 2.48 MPa.

@ full $v_c = .2\sqrt{f'_c}$ ≈ 1.18 MPa

∴ $A_v/s \geq$ 867 mm²/m @ f_y = 300

ie D12 - 130

cf - D12 - 400 + mesh ?

require $V_{wall} = 1.5 \times 1314 = 1971$ kN.

$\Rightarrow v_i \approx 2.45$ MPa over .8 lwal

$\Rightarrow v_c = .6\sqrt{P_e/A_g} \approx .62$ MPa.

$\therefore A_v/s \geq \frac{2.45 - .62}{410} \times 300 = 1339$ mm²/m.

$\rightarrow$ have H16s - 200 ef - $A/s = 2011$ mm²/m.

o.k.

Other walls all get H12s - 200

1) $\phi M_{code} = 9629$

$\therefore \phi V_{code} = 9629/6 \times 20.12 = 798$ kN

$\therefore V_{wall} \geq 1196$ kN $\therefore v_i = 1.92$ MPa.

@ $.6\sqrt{P_e/A_g} = .704$ MPa

$A_v/s \geq 890$ mm²/m - have 1131 mm²/m o.k.

Check confining.

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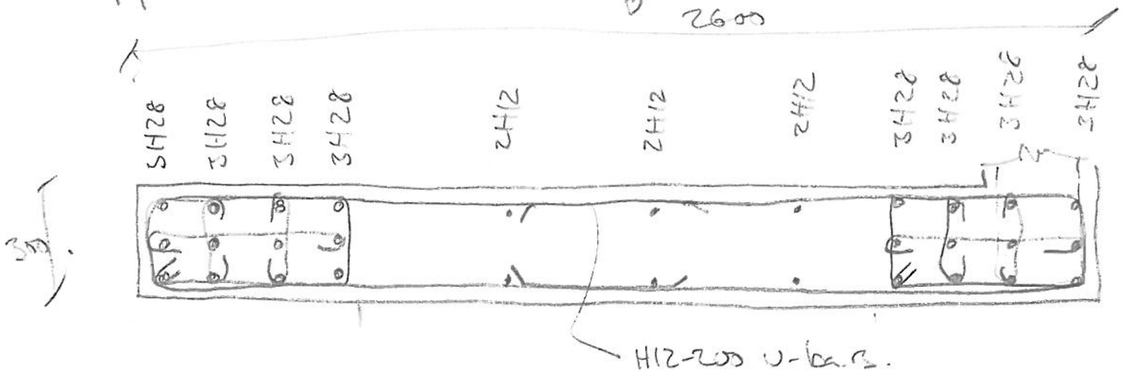
Wall capacity ✓

Appear to have steel ⇒



⇒ Wall supports not much more than self wgt at minimum, + nominal floor area.

∴ - self wgt @ base = 399 kN

∴ take $P_D = 450$ kN. $P_L = 40$ kN.

⇒ Analyse for min strength & overstrength @ $f_y = 410$ & $1.4 \times 380 = 532$ MPa.

3H28 - 1847 mm² @ 65, 215, 365, 515, 2085, 2235, 2385, 2535

2H12 - 226 mm² @ 910, 1300, 1690

use $f'_c = 35$ MPa.

⇒ @ $P_{0.9D} = 405$ kN - $M_i = 7451$ kNm

$P_{D+L} = 490$ kN - $M_i \phi = 9629$ kNm.

c = 381

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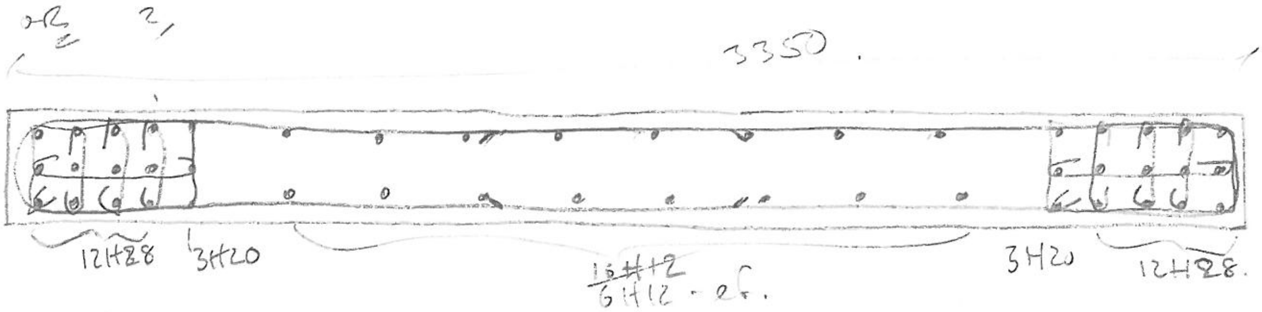

⇒ bar size & locations

1847	65
"	165
"	265
"	365
943	465
226	735
"	1000
"	1270
"	1540
"	1810
"	2080
"	2350
"	2615
943	2885
1847	2985
"	3085
"	3185
"	3285

A = ~~1350~~ 1356 @ 1675

⇒ with $P_D \simeq 750$ kN self wgt + 175 kN floor

$P_L = 150$ kN floor.

⇒ $M_i \simeq 12432$ kNm @ $P_{.9D} = 825$ kN

$M_i^{o} = 15873$ kNm. @ $P_{D+L} = 1075$ kN.

c = 503.

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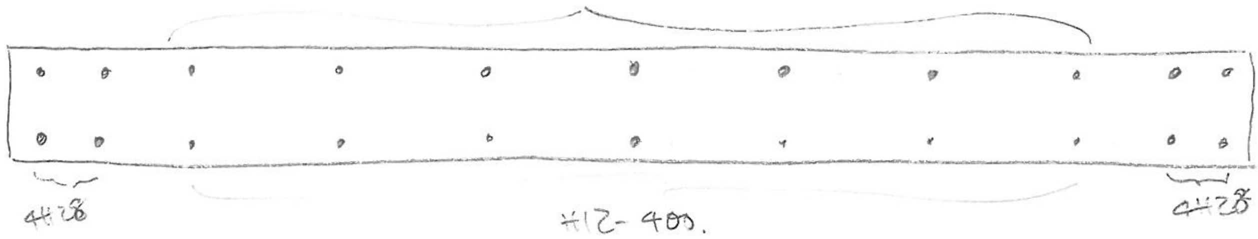
SR 3



⇒ 1232 @ 65, 215, 3135, 3285,

226 @ 475, 875, 1275, 1675, 2075, 2475, 2875

⇒ with P_D, P_L as 2,

⇒ M_i = 5474 kNm @ $P_{1.0D}$ = 833

$M_i \phi$ = 7000 kNm @ P_{D+L} = 1075

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Total capacity of walls

$\Sigma \phi M_i = .9(5474 \times 2 + 12432 + 7451)$
$= 27748$ kNm.

cf calculated bldg OTM = 37000 kNm.
ie have 75 % @ full Vstat
allowing for Dynamic analysis reduction
to .9 Vstat
+ over estimation $\approx$ 83% of .9 Vstat
~close.

Torsions to be resisted on other walls.

@ $\Sigma M_i^{o} = (2 \times 7000 + 9629 + 15873)$
$= 39502$
$\to \phi_o \approx 39502/27748 \approx 1.42$ ~ as expected

$\to$ allowing for reduced ~~Vstat~~ base.
such that $V_{\cancel{stat}\ base} \approx 25000/.6 \times 20.12$
$= 2070$ kN.

$\therefore V_i^{o} \approx 1.42 \times 2070 = 2939$ kN.

alternately, on 2, - $M_i^{o} = 15873$ kNm
implying ϕV_i code $\approx 15873/.6 \times 20.12 = 1314$ kN.
with $\omega_v \approx 1.5$ @ 6 storeys

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Recheck diaphragm.

→ @ V = 300 kN @ fully scaled value.
~~Vstorey~~ · 517 ~~x~~ · 6

⇒ with 8 D12 - 400

∴ available V_s @ 300 MPa.

ϕV_{si} 271 kN.

(→ if 2 layers 664 mesh are included

→ $\Delta \phi V_{si}$ = 385 kN.)

Require → with $\phi \omega_v \approx 2.5$

→ $V_i \phi \approx 750$

∴ @ $f'c$ = 25 MPa − $v_c = .2\sqrt{f'c} = 1.0$ MPa.

cf $v_i = 750 / .8 \times 2.6 \times .2 = 1.8$ MPa.

∴ $A_v/s \geq .8 \times 200 / 300 = 533$ mm²/m

⇒ D12s + 664 × 2 = 493 mm²/m − close

but have a problem where D12s stop before hitting Hibond.

In other direction, all walls.

∴ Take ϕV_i = 300/.6 × 2.5 = 1250 kN.

⇒ require tension steel for this.

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∴ over 11.5 m interface with floor

- $A_s \geq 1250/11.5 \times 300 = 362$ mm²/m.

⇒ with 664 mesh ~ 269 mm²/m.
short.

- Individually
- wall Line D/E from calcs $V_{code} = 349$ kN
⇒ $V_i^{\phi} = 2.5 \times 349 = 873$ kN.
∴ using distn calculated,
implies $V_{storey\, i}^{\phi}{}_{max} \approx 263$ kN.

- Would require $A_t \geq 263/300 = 876$ mm²
- have no steel. - maybe 2-H12
T = 93 kN.

Similarly, wall @ C
- calc $M_i^{\phi} = 16278$ (from DH calc's)
∴ $V_i^{\phi} \approx 16278/37683 \times 897$
= 387 kN.

⇒ Transfer in shear over $l = 3000$

- have ~10-D12 - $V_s = 339$ kN
- o.k.

Walls between.

similar loads but 1 wall takes end of beam - appears o.k. Other cube slab, but no steel shown. - possibly 2-H12 ?

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Summary

- Line 1 (South wall) E-W eq.
  - Probably o.k.

Line 4+ (North wall) E-W eq.
→ marginal

Line C (West toilet wall) N/S
o.k.

Line C-D (East wall toilet (West stair) N/S
- probably o.k.

Line D (Lift shaft/Stair wall) N/S
no steel shown. - or not much

Line D/E (East Lift shaft) N/S.
- no steel?

Entire shear core slightly dubious.

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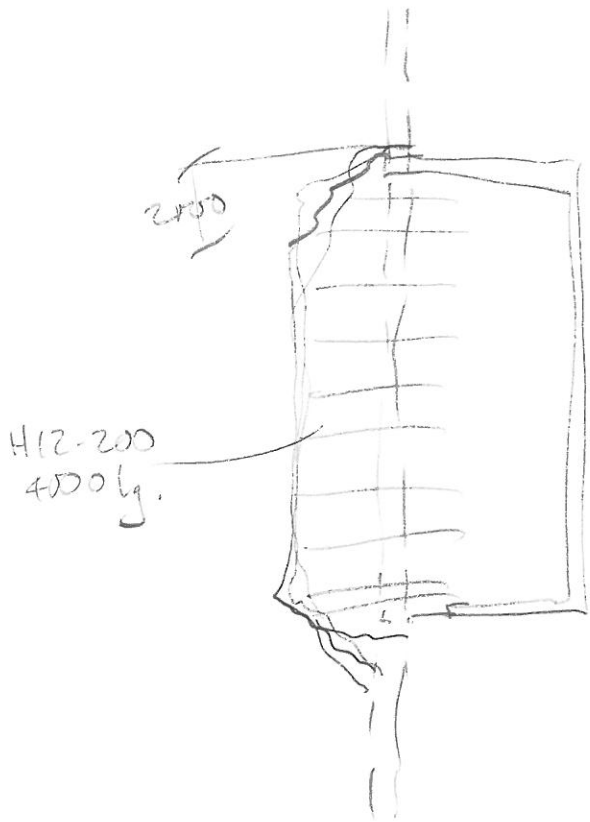
On extreme shear core



Cutting as shown. → get 4000 H12-200

$- V_s = 410 \times 4 \times 565 = 927$ kN

+ 664 mesh over 11.5 m.

$V_s = 11.5 \times 269 \times 275 = 851$ kN

- plenty already.

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Check roof

$h = 21$ m — say 20 m.

$\therefore V = 40$, $S_1 = 1.0$, $S_2 = .9$

$\therefore V_s = 36$ m/s $\therefore q = .794$ kPa.

$\therefore$ @ $C_{pi} = \pm 0.3$, $C_{pe} = -1.0$

$\therefore p_w = 1.03$ kPa.

– on 7.5 x 7 grid

$\Rightarrow l = 7000$, trib width = 7500

$\Rightarrow W_w = 7.73$ kN/m.

178 x 89 RSJ $\therefore \delta_w = 16.2$ mm $= .0023L$ @ full continuity

$\therefore$ end span – $= 33.7$ mm
$= .0048L$ – high.

$\therefore$ would expect some problems with tiles.

$\Rightarrow$ on 5-yr wind – drop q to .541 kPa

$\therefore W_{w_5} = 5.27$ kN/m $= 11.1$ mm mid.

$\therefore$ endspan $= 23.0$ mm

$= .0033L$ – still a little high.

assume strength o.k. – check. @ $p_L = .35$ kPa.

$\Rightarrow$ max $M.7D+W = .36$ kNm. $wl^2/8$

$\cancel{M_{ATS}} = 39$ kNm.

cf M_{xx} max $= 28.9$ kNm $\Rightarrow wl^2/12$ approx

→ end spans - 4.75 m.

∴ $wl^2/_{2} = +16.7$ kNm   W.70-m = - 5.89

OR internal $wl^2/_{12} = +24.1$ kNm.

- over l = 1400

- 12.6 over l = 5000.

o.k.

cf WDTS end = -18.0

int = -26.0 - over l = 1600 - just o.k

+ 13 -

o.k @ capacity = 28.9 kNm. fr.

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Ala Reay 660 434.

9, 11, 14, 15, 16, 17, 19,
25, 26, 30, 31, 32,
33, 36

→ Come in any time.