In the matter of the Commissions of Inquiry Act 1908

And

In the matter of the Canterbury Earthquakes Royal Commission

Brief of evidence of Henry John Hare relating to the CTV Building (249 Madras Street)

Date: 1st June 2012

DLA PHILLIPS FOX

209 Queen Street, Auckland 1010
PO Box 160, Auckland 1140
DX CP24027 Auckland
Tel +64 9 303 2019
Fax +64 9 303 2311

Solicitor on the record Grant Macdonald grant.macdonald@dlapf.com Tel +64 9 300 3896
Contact solicitor John Hannan john.hannan@dlapf.com Tel +64 9 300 3843

611263_1

Qualifications and Experience

1 My full name is Henry John Hare.

2 I am a Director of Holmes Consulting Group Limited ('**HCG**'). I have a Bachelor of Engineering (Civil) with Honours, and am a Chartered Professional Engineer. I hold professional memberships with the Institution of Professional Engineers, the Structural Engineering Society (of which I am the current President) and the New Zealand Society of Earthquake Engineering. In addition I am a licensed Professional Engineer in California.

3 I have over 25 years of experience in structural engineering in New Zealand, England, Hong Kong and the United States, where I was resident from 2000 to 2005.

4 I spent over 4 years resident in California from 2000 to 2004. Previously I had worked on assessments of buildings after the 1994 Northridge earthquake. Whilst resident, I took the additional examinations required to be licensed in California. This included a seismic specific examination. Much of the work I carried out involved retrofitting and strengthening buildings to upgrade their seismic capacity.

5 The majority of my professional career has been with HCG, where I have worked at various times in Auckland, New Plymouth and Christchurch. My project experience has been mainly in buildings, with a combination of both new building design and evaluation and strengthening of existing buildings. I am currently seconded for the majority of my time to CERA where I am the acting Principal Engineering Adviser.

Scope of Evidence

6 This brief of evidence sets out my involvement, whilst employed by HCG, in the preparation of report for a prospective purchaser of an office development located at 249 Madras Street, subsequently known as the CTV Building (**the CTV Building**).

Background

7 On 24 January 1990, HCG was engaged by Buddle Findlay Limited and Schulz Knight Consultants Limited to prepare a pre-purchase report on the CTV Building. HCG was advised that Buddle Findlay Limited and Schulz Knight Consultants Limited were acting on behalf of a potential purchaser of the CTV Building.

8 Initially, HCG was unaware of the identity of the potential purchaser. HCG subsequently became aware that the potential purchaser was the Canterbury Regional Council (**CRC**).

9 At the request of our client, HCG's review of the CTV Building included a site inspection of all areas that were readily accessible, a review of available documents and carrying out approximate calculations. The report was intended for a potential purchaser. The timeframe given by the purchaser was limited. So the report was never intended to be a full peer review of the design of the building. The Draft Report subsequently produced by HCG confirmed that "due to the limited time available for the report, our review has been limited...." **[BUI.MAD249.0005.11].**

10 I drafted the report and carried out the approximate calculations. The calculations were approximate calculations in accordance with our brief, and because the client requested HCG stop work and produce a report based on the information HCG had collated at that time, as I will explain later in my evidence.

11 My work was supervised by Grant Wilkinson, who at the time was a Project Director for HCG and senior to me. My position at the time was Senior Engineer for HCG.

12 The draft report subsequently produced by HCG is dated January 1990 (**the HCG Draft Report**). [**BUI.MAD249.0005.7-16**].

Review of Structural Drawings and Calculations

13 Copies of the architectural drawings and some of the structural drawings were obtained from Alun Wilkie Architects, the architects for the building.

14 From these drawings, I was able to carry out an approximate seismic analysis on 25 January 1990. **[BUI.MAD249.0005.28-41].** In the course of preparing my evidence it has been brought to my attention that my calculations carried out on this date appear to be dated 25 September 1990, but they were definitely carried out in January 1990.

15 The relevant Standards at the time were:

15.1 the Loadings Standard NZS4203:1984 for derivation of the demand (loads); and

15.2 the Concrete Design Standard NZS3101:1982 for the design of the elements.

16 My first task was to review the primary load paths

17 Having reviewed the structural drawings and carried out initial calculations, I identified that there appeared to be an area of non-compliance with the code of the day with respect to the tying of the floors to the shear walls, specifically to the north core walls. I picked this up fairly quickly as there appeared to be no connection detailed for the walls on either side of the lift shaft.

18 My calculations noted the following:

18.1 Gridline 1 (South Wall) - 'probably OK';

18.2 Line 4+ (North Wall) - 'marginal';

18.3 Gridline C (west toilet wall) - 'OK';

18.4 Line C-D (east Wall toilet/West stair) - 'probably OK';

18.5 Line D (Lift shaft/stair well) - 'no steel shown or not much'; and

18.6 Gridline D/E (East lift shaft) - 'no steel?'.

19 In the summary of my approximate seismic analysis, I noted 'entire shear core slightly dubious'. This comment related to the connection of the north shear core as a whole to the floor diaphragms. The connections in particular seemed tenuous and in need of further verification.

20 The floor diaphragm of the CTV Building adjacent to the north shear core was punctured by the lift, stair and service risers. The result was that there were relatively few direct connections from the floor diaphragm to the north shear walls and there appeared to be insufficient reinforcement tying the floors and shear core together.

21 On 25 January 1990, I also telephoned the offices of Alan Reay Consultants Limited (**ARC**), the Structural Engineers for the building, to arrange for an inspection of their documentation. I believe I spoke to Alan Reay directly. Given the passage of time, I do not recall the content of the discussion in detail but I wanted to arrange to see ARC's documentation to check that the drawings I was working from were the most up to date, and to see whether a site instruction may have been given that would have dealt with the connection of the floor to the walls. My attendance note confirms that I was told I could inspect ARC's documents at any time.

22 In the course of preparing my evidence it has been brought to my attention that my attendance note of this telephone call appears to be dated 25 September 1990 **[BUI.MAD249.0005.42]**. I must have written the wrong date because I attended the ARC offices on 26 January 1990 and made a note of this attendance which is dated 26 January 1990 **[BUI.MAD249.0005.24-27]**.

23 When I attended the offices of ARC on 26 January 1990 I reviewed ARC's design documentation, soils investigation and a complete set of drawings. I cannot recall who met me when I arrived at ARC's offices but I recall seeing both Alan Reay and Geoff Banks at some stage during my visit. I believe I discussed my concern in relation to the floor diaphragms with either Alan Reay or Geoff Banks during my visit. ARC indicated that there may have been some provision made for this during construction and that enquiries would be made.

24 I recall someone at ARC suggesting they would use a reinforcement cover meter (or bar finder) to determine whether there was in fact some reinforcement in the locations in question.

25 I was informed by either Alan Reay or Geoff Banks, I cannot recall who, that the original design engineer was David Harding. He was unavailable for comment as he had left ARC but I was told that Geoff Banks was available for comment on aspects of the design.

26 I conducted a review of the documents and made notes on matters of interest from the calculations and drawings whilst at the office of ARC. These notes are on my attendance note. **[BUI.MAD249.0005.24-27]**.

27 The ARC design calculations included two pages (Pages S56 and S57) which considered the "Slab Diaphragm & connection to shear walls". The ARC calculations failed to address the tie force to the walls in question (although they did consider the shear calculations for the orthogonal walls, including the south wall, and the north wall through the shear in the slab). That is, the calculations on this aspect of the design addressed earthquake in the east-west direction only. The calculations for this aspect of the design appeared to omit consideration of an earthquake in the north-south direction.

Bryan Bluck of Christchurch City Council

28 On 29 January 1990, I spoke with Bryan Bluck, who was then the Buildings Control Manager at Christchurch City Council. My attendance note suggests I contacted him at approximately 2.15pm that day **[BUI.MAD249.0005.23]**. The purpose of the call was to enquire whether the Council had identified any issues during the building permit and construction process of the building.

29 I mainly spoke to Bryan Bluck about issues concerning the egress stair which was hanging over the boundary. I do not recall any structural issues being raised nor do I recall any concerns being raised about the building permit process.

30 I do not recall discussing the diaphragm/shear wall connections with Mr Bluck. I suspect the reason this wasn't raised by me was because ARC had previously suggested that there might be some ties, and this was to be verified with the use of the bar-finder. At that time, I did not know if I had been supplied with the most recent drawings. Updated drawings might have addressed the diaphragm/shear wall connection position. Also, I did not know whether the position had been addressed during the construction phase. This was still being investigated by ARC.

Inspection of Building

31 I inspected the CTV Building on 30 January 1990 and was met on site by Geoff Banks. He brought a bar finder with him, which he had hired, so as to determine whether reinforcement had in fact been added during the construction process. Apparently ARC had been unable to locate any documentation to confirm whether provision had been made for the floor diaphragm attachment issue during construction.

32 Geoff Banks tested the slab in the areas concerned with the bar finder. I did not stay with Geoff Banks throughout the testing. However, whilst I was with him no significant reinforcement was found.

33 The HCG Draft Report records that Level 1 and Level 4 were unavailable for inspection. These Levels were occupied at the time. The remaining floors were accessible and were taken as representative. Geoff Banks and I were also able to gain access to the lift machine room, the cooling tower and onto the roof.

HCG's Draft Report sent to Schulz Knight on 31 January 1990

34 On or about 31 January 1990, HCG was asked to supply a copy of HCG's report as it stood at that time to our client's representative. I cannot recall how this request was communicated but assume the request was made through Grant Wilkinson by Robin Schultz of Schulz Knight Consultants Limited. A draft report was produced, based on the information that had been collated to that date. I faxed the HCG Draft Report to Robin Schulz of Schulz Knight Consultants Limited on 31

January 1990. The fax cover sheet confirms I was sending "a draft copy of our report" **[BUI.MAD249.0005.1]**.

35 At paragraph 3.0, the HCG Draft Report stated:

> 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.

36 At paragraph 6.3, the HCG Draft Report stated:

> 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.

Floor connections

37 My view was that it appeared that more reinforcing or other connection was required in those areas of the floors that connected to the north shear core along gridlines D and D/E.

38 I communicated this issue to ARC at the time of my first visit to ARC's offices. It was later confirmed by their own testing on site. Our understanding (which was confirmed by the letter from ARC dated 2 February 1990) was that ARC had acknowledged there was an issue and it was taking the necessary steps to resolve the issue. Any modification to the design of the building would have had to have been approved by the Council through the permitting process.

HCG's Preliminary Remedial Detail and Estimate for Costs of Remedial work

39 The CRC, or at least its representatives, had asked HCG to advise on the likely cost of fixing any defects we found. I cannot recall how or when CRC requested advice on the likely cost of fixing any defects in the CTV

Building as identified in the HCG Draft Report. It may have been a verbal request from Warren and Mahoney (who were acting as the principal consultant between HCG and Schulz Knight Consultants Limited).

40 I developed a possible remedial detail on 31 January 1990, to give a preliminary estimate of the cost of establishing a connection between the north core and the floors. **[BUI.MAD249.0005.19-20].**

41 My draft remedial detail involved:

41.1 The insertion of steel diaphragm strengthening members (drag bars) on Levels 1 to 5 of the CTV Building(by Levels 1 to 5, I was referring to all levels above the ground floor car park. In my calculations and in the HCG Draft Report, HCG refers to the floors in the CTV Building as the Ground floor and then levels 1 to 5 above the Ground floor. In contrast, ARC refers to the Ground floor as Level 1 and the floors above the Ground floor as Levels 2 to 6).

41.2 A piece of steel angle being cut to shape and fixed to the lift walls and the underside of the slab, with drilled and epoxied anchors. There were to be two such ties per floor, to the wall on grid D and the east wall part way between grids D and E. In my view, such a detail was required to improve the connection between the floor slabs and the walls of the North core.

41.3 Consideration of the specifics of fixing to a thin wall and a thin slab. I recommended the use of 12mm diameter mild steel rods for connection to the wall, being the largest diameter reinforcement that could be practically developed into the 300mm wall. Likewise, I limited the size of the connections into the underside of the slab, and recommended coring and grouting from above, due to concerns I had with overhead epoxying and grouting.

42 My draft remedial detail was developed solely for the purpose of determining the approximate cost of any remedial work, as part of HCG's

report to the prospective purchaser. My draft remedial detail was not developed to a final design stage.

43 On or about 31 January 1990, HCG was instructed to cease any further work and our engagement ceased. I think HCG was advised of the identity of the potential purchaser at the time we were asked to supply our draft report.

44 On 1 February 1990, Grant Wilkinson sent a memo to Kerry Mason of Warren & Mahoney with details of the quote we had received for the proposed remedial works **[BUI.MAD249.0005.17-20]**. I supplied the sketch for the quote but don't remember any further involvement beyond that.

45 The quote came from Martin Charles, a quantity surveyor with Russell Drysdale and Thomas. He advised that the cost to carry out the HCG draft remedial detail would be approximately $14,000 plus GST. In his memo to Warren & Mahoney, Grant Wilkinson asked Kerry Mason "do you need anything else from us on this job?". I do not know whether Kerry Mason had any further contact with Grant Wilkinson about the building after this.

46 HCG was subsequently advised that CRC had decided not to proceed with the potential purchase of the CTV Building. I don't know exactly when we were told this or why the CRC had decided not to proceed with the purchase.

47 In the documentation that has subsequently been made available through the Commission's process, I have been made aware of a letter from ARC to its insurers, dated 1 February 1990In the letter, ARC says that CRC has an option to purchase until 28 February 1990 and that CRC's solicitor has requested a 2 month delay in settlement to give time to carry out the remedial work. Refer to **[BUI.MAD249.0129.2-3]**. I was not aware in January or February 1990 of any "option to purchase" or any request for a delay in settlement.

Further discussion with ARC on 14 February 1990

48 Geoff Banks faxed a letter to Grant Wilkinson dated 2 February 1990 in which ARC set out their understanding of the scope of the "possible non-compliance" referred to in the HCG Draft Report **[BUI.MAD249.0005.5].** In the letter, ARC suggested:

48.1 "the scope of the possible non-compliance" related to "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)";

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

48.3 "the agreed maximum tie load is 300kN 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".

49 HCG was asked to contact the office of ARC "today" if HCG's understanding of the situation was not as outlined in the letter.

50 I cannot recall whether I was shown this letter at the time. I have seen it whilst preparing my evidence for the Commission. I can't recall whether I discussed the content of the letter with Grant Wilkinson and/or ARC.

51 Geoff Banks called me by telephone on 14 February 1990. He wanted to discuss the design issue with me and I had a short discussion with him. I did this without a fee being rendered.

52 I believe I discussed the content of the telephone discussion afterwards with Grant Wilkinson.

53 In the documentation that has subsequently been made available through the Commission's process, it appears ARC was told by the receivers, KPMG Peat Marwick, by letter dated 2 February 1990 to agree the level of the work required. **[BUI.MAD249.0129.27-28]**. It also appears that ARC was given approval by its insurers, by letter dated 12

February 1990, to agree the precise scope of the work HCG considered to be inadequate. **[BUI.MAD249.0129.29]**. As far as I am aware, none of this was communicated to HCG at the time.

54 I have recently been shown a copy of the file note made by Geoff Banks dated 14 February 1990 **[BUI.MAD249.0130.14]**.

55 The file note suggests we:

55.1 "agreed loads" applicable to each level of the CTV Building, as follows:

55.1.1 Level 5 300 kN

55.1.2 Level 4 240 kN

55.1.3 Levels 1,2,3 184 kN

55.2 "confirmed tie only system ok", but the note makes reference to "possible pretension via nuts".

55.3 "confirmed reduced connection at L1 may be ok (could compensate at L2 if necessary)".

56 However, my recollection differs from Geoff Banks's recollection of the matters discussed during this telephone discussion. I recall that we discussed:

56.1 the loads referred to in his note, in the context of Geoff indicating he had carried out his own calculations and had arrived at these figures. I indicated that they appeared to be around the right figures but it was over to Geoff to check and finalise.

56.2 Geoff's suggestion that the loads could be reduced at any level in light of his suggestion that some steel existed at the applicable level. I indicated that if this were the case, then the loads could potentially be reduced but it was over to Geoff to check and finalise.

56.3 Geoff's suggestion that he use a reduced connection at Level 1 which was to be compensated for at Level 2, if necessary. I indicated that Geoff would have to check whether this was possible by investigating other mechanisms that would be required to make up any shortfall. I expressed the view that caution should be exercised if it were ARC's intention to reduce the load at Level 1. I certainly did not agree to this.

57 The discussion was a general discussion and centred entirely on the Loadings Standard NZS4203:1984 for derivation of the demand (loads).

58 I wouldn't describe us reaching agreement. In relation to all matters, it was clearly understood that it was over to ARC to progress.

59 The file note of the telephone discussion as recorded by Geoff Banks says "will confirm if work proceeds". I do not recall any suggestion during our telephone discussion that ARC was to confirm anything with me. HCG thought its involvement was over. We assumed ARC was sorting out the design issue and I did not expect to hear from Geoff Banks again.

60 Following my discussion with Geoff Banks, my understanding was still that ARC had accepted responsibility for dealing with the matter and that they intended to take the necessary steps to remedy the issue. I had no further involvement with the CTV Building post February 1990.

ARC Remedial Detail

61 I understand that Alan Reay and Geoff Banks may now be asserting that HCG had some level of responsibility for the ARC design of the remedial work that ARC ultimately carried out for the CTV Building. This is not correct.

62 HCG was engaged by a prospective purchaser to prepare a pre purchase report. I contacted ARC and notified them of the potential problem I had

identified during that process. I also discussed the issue with Geoff Banks. I had not carried out detailed calculations nor had I finalised the design for the Draft HCG Remedial Detail. I was not instructed to do so. I understood that ARC, as the designer of the building had accepted sole responsibility for rectification of the issue.

63 The ARC remedial work which appears to have been carried out to the CTV Building by ARC in 1991 did not, in any event, accord with the Draft HCG Remedial Detail which I had produced for the prospective purchaser of the CTV Building.

64 I did not give ARC a copy of the drawings or calculations for that Draft detail or a copy of the Draft HCG Report.

65 I have been requested by Counsel assisting the Commission to review ARC's calculations in respect of the ARC remedial work which was designed by ARC and carried out to the CTV Building under ARC's supervision. **[BUI.MAD249.0130.15-27].**

66 It appears that initial calculations were carried out by ARC on 29 January 1990 and 1 February 1990. Further calculations appear to have been carried out on 10 October 1991 (some 20 months after my discussions with Geoff Banks). **[BUI.MAD249.0130.21-27].**

67 The calculations dated 10 October 1991 include calculations (at pages 12A-14A) for the transfer of loads from Levels 1 and 2. It appears those calculations were carried out by ARC to "check whether loads in diaphragms at these levels can be transferred to walls 5 and 7 (lines C and C/D)". The ARC calculations conclude that the additional load on the other walls (lines C and C/D) would be acceptable as those walls had excess capacity. But the calculations do not appear to include any additional analysis of the building's global performance.

68 The calculations do not appear to have considered the impact of any torsional behaviour at Levels 1 and 2. Without specific analysis, this would be difficult to estimate, but there is a possibility that there could have been additional rotation of the floor plate due to the eccentricity of

the connection. By this I mean the whole of the floor plate appears to have been left eccentrically connected to the wall system on the north side. Therefore the floors that were not connected could tend to behave differently in the event of shaking, potentially imposing greater displacements to the gravity structure, and greater stresses in the connecting elements at the level above. A detailed computer analysis would probably have been required to verify the impact of this.

69 The ARC remedial detail as designed by ARC adopted the same concept (a tie system) that I had proposed in the Draft HCG Remedial Detail for the prospective purchasers of the CTV Building. However, the ARC remedial detail differed from the Draft HCG Remedial Detail in several ways.

70 I have identified some of the differences as follows:

70.1 ARC used a 152x152x10 angle. The Draft HCG detail recommended a 200x200x10 angle.

70.2 ARC used 262kN as a maximum load. The Draft HCG detail recommended 300kN.

70.3 ARC used a ratio of area supplied to area required of 1.33 times (allowing for bolt holes). The Draft HCG detail recommended a ratio of 1.43 times the area required.

70.4 ARC used fewer, but larger, connections, comprising M24 Chemsets to the wall, with M20 Chemsets to the underside of the slab. The Draft HCG detail recommended smaller connections to ensure the anchors had sufficient anchorage to develop the full capacity of the anchors, given the thickness of the elements being fixed to. The Draft HCG detail recommended epoxied bars rather than Chemsets as I had concerns about the Chemset adhesive (which I felt was best used on light-duty applications).

70.5 The Draft HCG detail recommended coring and grouting from above to attach the angles to the floors, due to concerns with overhead epoxying and grouting. I was not confident at the time that Chemset anchors were suitable for overhead application, having had several experiences of Chemsets not curing properly and of the Chemset adhesive running out of horizontal holes.

70.6 The slab thickness was 200mm overall but with Hi-bond, taking into account the trough depth of 50mm, this left 150mm. The slab thickness was less than the minimum acceptable depth of fixing at the time for M20 anchors, therefore prone to pull-out failure. To address this, the Draft HCG detail recommended a headed anchor instead. The hole was to be cored from above over the entire depth, the sides roughened, and then filled with non-shrink high-strength grout. In this way the detail would have approximated a conventional in situ connection.

70.7 The angle used by ARC was slightly shorter, with the spacing of bolts adjusted to suit. The Draft HCG detail recommended the angle be extended as far as possible in order to connect into as great an area of slab as possible. I had not attempted to minimise its length to suit minimum anchor spacings.

70.8 ARC inserted drag bars at the top 3 Levels only. The Draft HCG detail recommended drag bars at each floor level above the ground floor.

71 The Draft HCG Remedial Detail was very different to the ARC remedial detail. For the reasons as set out above, I would not have agreed to the ARC Remedial Detail had I been consulted about it at the time.

72 The Remedial Detail used in the CTV Building was designed by ARC without any reference to HCG and was never approved by HCG. Geoff Banks, on ARC's behalf, carried out independent calculations. ARC's decision to leave out the ties at Levels 1 and 2 was a decision taken by ARC long after my discussions with Geoff Banks.

73 In any routine review situation, the original designer remains responsible for the design. HCG identified a potential design issue and this was referred back to ARC, the original designer. Any required modification to the design of the building would have had to have been referred by ARC to the Council for approval through the permitting process.

74 As a result of discussions with ARC and ARC's letter dated 2 February 1990, HCG had a clear understanding that ARC had accepted responsibility for the design issue and was taking steps to remedy it. HCG was satisfied that that is what ARC intended to do. That was the end of the matter as far as HCG was aware.

Date: 1st June 2012

....
Henry John Hare

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.

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

CALCULATIONS

JOB NAME 249 Madras St
JOB No W8165
CALCS BY
PAGE
DATE 25/4/90

APPROX SEISMIC ANALYSIS.

Floors all typical

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

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

ℓbeams = 74.5 m 400 × 550(350)

+ 60.5 m 960 × 550(350)

ℓwalls = 26m @ 300 ← Coupled.
+ 4m @ 400.

columns = 20 + lightweight wall ℓ = 93m.

∴ 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 = 26 A Level 7 = 7 × 4.5 (150 Hibond) + 300 walls P_D = 600 kN

h = 7.4 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.

WIT.HARE.0001.29

BUI.MAD249.0005.29

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

CALCULATIONS

JOB NAME PAGE
JOB No CALCS BY DATE

∴ Lave & Wt = 5(5057 + 605)
+ (600 + 53) + (700 + 86)
= 29750 kN

@ Cd = CKSR = .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

$S_p = 1.0$ ∴ $C_{p\,max} = 0.3$

∴ $C_p = K\, \underbrace{K_{hz}}\, Z\, R\, C_{p\,max}$

= 0.76 × 5/6 × 1.0 × 0.3

= .19.

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

CALCULATIONS

JOB NAME PAGE
JOB No CALCS BY DATE

∴ @ .6 Vstorey to wall $-V = .19 \times .6 \times 5662$
$= 645$ kN.

⇒ this load must go via small section
or slab = 2600 long × 200 thick
⇒ $V_i = 645/.2 \times 2.6 = 1.24$ 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
∴ $A_v/s \geq 1554$ mm²/m.
ie D12s – 70 crs.
not 400 !

OR, ⇒ say overstrength = 2.0 max
⇒ $V_i = 2 \times 645/.2 \times 2.6 = 2.48$ MPa.
@ OJII $V_c = .2\sqrt{f'_c} \approx 1.8$ MPa
∴ $A_v/s \geq 867$ mm²/m @ $f_y = 300$
ie D12 – 130
cf - D12-400 mesh ?

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

CALCULATIONS

JOB NAME PAGE
JOB No CALCS BY DATE

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

$\Rightarrow v_i \simeq 2.45$ MPa over .8 Lwall

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

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

→ have H16s - 200 ef — $A/s = 2011$ mm²/m.
o.k.

Other walls all get H12s - 200

1) ~~→ case~~ $\phi M_{code} = 9629$

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

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

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

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

.. each confining.

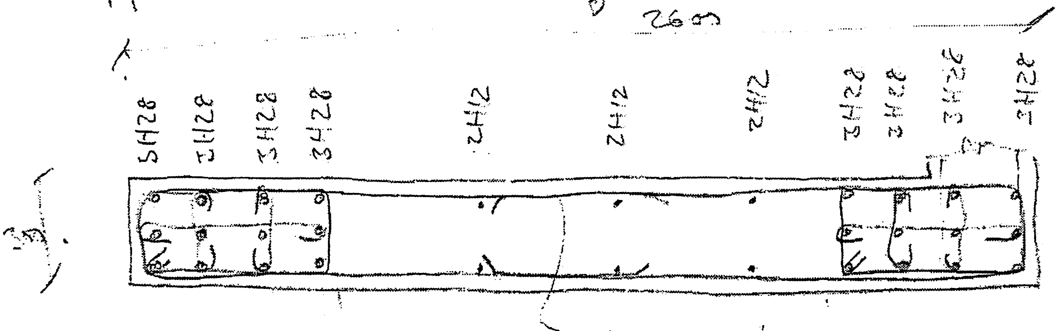
HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

CALCULATIONS

JOB NAME PAGE
JOB No CALCS BY DATE

Wall capacity 1/

Appear to have steel of



⇒ 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 & 1.4×380 = 532 MPa.

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

2H12 - 226 mm² @ 910, 1300, 1690

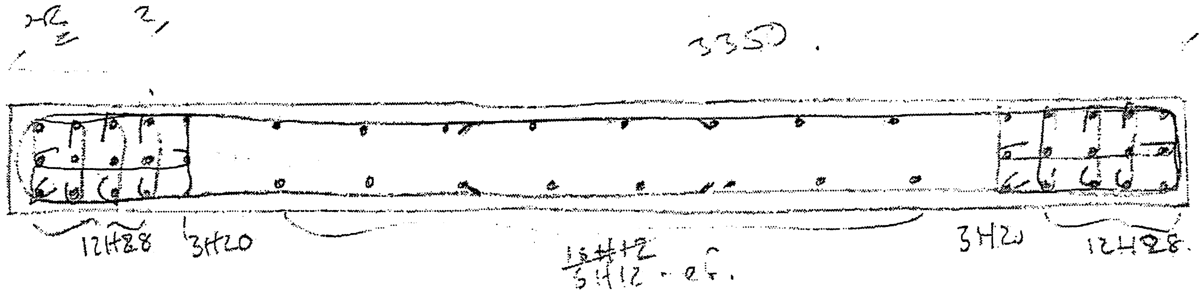
use f'_c = 35 MPa.

⇒ @ P_{DL} = 405 kN - M_i = 7451 kNm

P_{DL} = 490 kN - $M_i \phi$ = 9629 kNm.
c = 381

CALCULATIONS

JOB NAME PAGE
JOB No CALCS BY DATE



⇒ 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

$A = \cancel{1288}$ 1355 @ 1675

⇒ with P_D = 750 kN self wgt + 175 kN floor
P_L = 150 kN floor.

⇒ M_c = 12432 kNm @ $P_{.9D}$ = 833 kN
$M_c{}^d$ = 15873 kNm. @ P_{D+L} = 1075 kN.
c = 533.

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

CALCULATIONS

JOB NAME

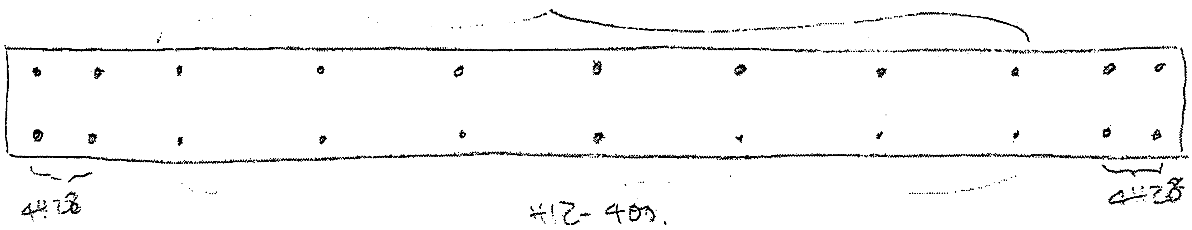
PAGE

JOB No

CALCS BY

DATE

STC 3



= 1232 @ 65, 215, 3135, 3285.

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

⇒ with P_D, P_L as 21

⇒ M_i = 5474 kNm @ P_{40} = 833

M_i^f = 7000 kNm @ P_{D+L} = 1075

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

CALCULATIONS

JOB NAME
JOB No
CALCS BY
PAGE
DATE

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 V_{stat}

allowing for Dynamic analysis reduction to $.9 V_{stat}$

+ over estimation = 83 % of $.9 V_{stat}$

~ close.

Torsions to be resisted on other walls.

@ $\Sigma M_i^{o} = (2 \times 7000 + 9629 + 15873)$

$= 39502$

→ $\phi_o = 39502 / 27748 = 1.42$ ~ as expected

→ allowing for reduced ~~Vstat~~ base.

such that $V_{base} = 25000 / .6 \times 20.12$

$= 2070$ kN.

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

alternately, on 2, - $M_i^{o} = 15873$ kNm

implying ϕV_i code $= 15873 / .6 \times 20.12 = 1314$ kN.

with $\omega_v = 1.5$ @ 5 storeys

Recheck diaphragm

→ @ V = 300 kN @ fully scaled value.
Vfamax · 517 × ·60

⇒ with 2 D12-400

∴ available V_c @ 300 mm.

ϕV_{SC} 271 kN.

(→ if 2 layers 664 each are included
→ $\Delta \phi V_{SC}$: 385 kN.)

Require → with $\phi \omega_v$ ≈ 2.5

→ $V_i \phi$ ≈ 750

∴ @ f'c = 25 MPa – v_c = $\cdot 2\sqrt{f'_c}$ = 1·0 MPa.

cf v_i = 750 / ·8 × 2·6 × ·2 = 1·8 MPa.

∴ A_v/s ≈ ·8 × 200 / 500 = 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.

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

CALCULATIONS

JOB NAME PAGE
JOB No CALCS BY DATE

∴ over 11.5m interface with floor
– $A_s = 1250/11.5 \times 300 = 362$ mm²/m.
⇒ with 564 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 $V_{diaphragm}^{\phi}{}_{max} = 263$ kN.

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

Similarly, wall @ C
– calc $M_i^{\phi} = 16278$ (from LH calcs)
∴ $V_i^{\phi} = 16278/37683 \times 897$
= 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 over slab, but no steel shown. – possibly 2-H12 ?

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

CALCULATIONS

JOB NAME PAGE
JOB No CALCS BY DATE

Summary

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

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

Line C (West toilet wall) N/S
ok

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

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 deficient.

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

CALCULATIONS

JOB NAME

PAGE

JOB No

CALCS BY

DATE

On external 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.

HOLMES CONSULTING GROUP

STRUCTURAL AND CIVIL ENGINEERS

Offices in Christchurch, Wellington, New Plymouth, Auckland.

CALCULATIONS

JOB NAME | PAGE
JOB No | CALCS BY | DATE

Check roof

$h = 21$ ~~m~~ say 20 m.

∴ $V = 40$, $S_1 = 1.0$, $S_2 = .9$

∴ $V_s = 36$ m/s ∴ $q = .794$ kPa.

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

∴ $p_w = 1.03$ kPa.

→ on 7.5 × 7 grid

∴ $l = 7000$, rib width - 7500

→ $W_w = 7.72$ kN/m.

178×89 RSJ ∴ $\delta_w = 16.2$ mm = .0023 L @ full continuity

∴ end span - = 33.3 mm
= .0048 L - high.

∴ would expect some problems with tiles.

→ on 5-yr wind - drop $q \div .541$ kPa

∴ $W_{w5} = 5.27$ kN/m = 11.1 m wid.

∴ endspan = 23.0 mm

= .0033 L - still a little high.

assume strength o.k. - check @ $p_s = .35$ kPa.

→ max $M_{.75+w} = -36$ kNm. $wl^2/8$

$M_{DFS} = 39$ kNm.

cf $M_{max} = 28.7$ kNm → $wl^2/12$ approx

HOLMES CONSULTING GROUP

STRUCTURAL AND CIVIL ENGINEERS

Offices in Christchurch, Wellington, New Plymouth, Auckland.

CALCULATIONS

JOB NAME | PAGE
JOB No | CALCS BY | DATE

→ end spans - 4.75m.

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

OR internal $wl^2/2$ = +24.1 kNm.

- over l = 1050

- 12.6 over l = 5000.

cf WLRS end = -18.0   o.k.

int = -26.0 over l = 1050 - just o.k

+13 -

o.k @ capacity = 28.9 kNm. for.

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

CALCULATIONS

JOB NAME

JOB No

CALCS BY

PAGE 8

DATE 25/6/90

Ala Reay 660 4-3e.

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

→ Come in any time.

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

MEMO

JOB NAME 249 Madras st.
JOB No

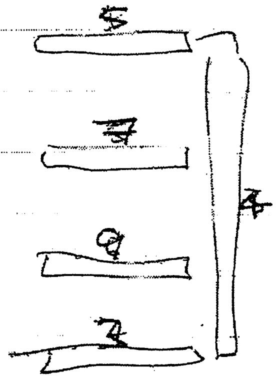
DATE 26/1/90

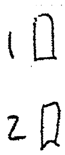
Documents @ Alan Reay.

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

Cd = .1 as calc.
– Stab analysis

Static $\Sigma V = 3300$ kN OTM = 47254 kNm.
$T_x = T_y = 1.06$ sec ∴ x.712 ∴ better.
Dynamic analysis performed.





1,2	P_D = 1493	P_L = 182
3	461	35
4	2101	
5	2073	234
7	1898	351
9	541	

COPIES TO

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

MEMO

JOB NAME

JOB No

DATE

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 ~~sed~~ incr.

T x .6 for wall @ edge = 300 kN max

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

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

MEMO

JOB NAME

JOB No

DATE

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 4²

Designed beams to SM-1-6

↳ not live 1 – used overstrength

~ seems ok

- Look @

COPIES TO

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

MEMO

JOB NAME

JOB No

DATE

Roof steel

178 x 73 RSJ @ 7.00 span
@ 7300 crs!

COPIES TO

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

MEMO

JOB NAME W8165

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 welds v. bad in corners. time will have construction faults.

- "Terrible elevation" - check vehicle clearance.

COPIES TO

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.

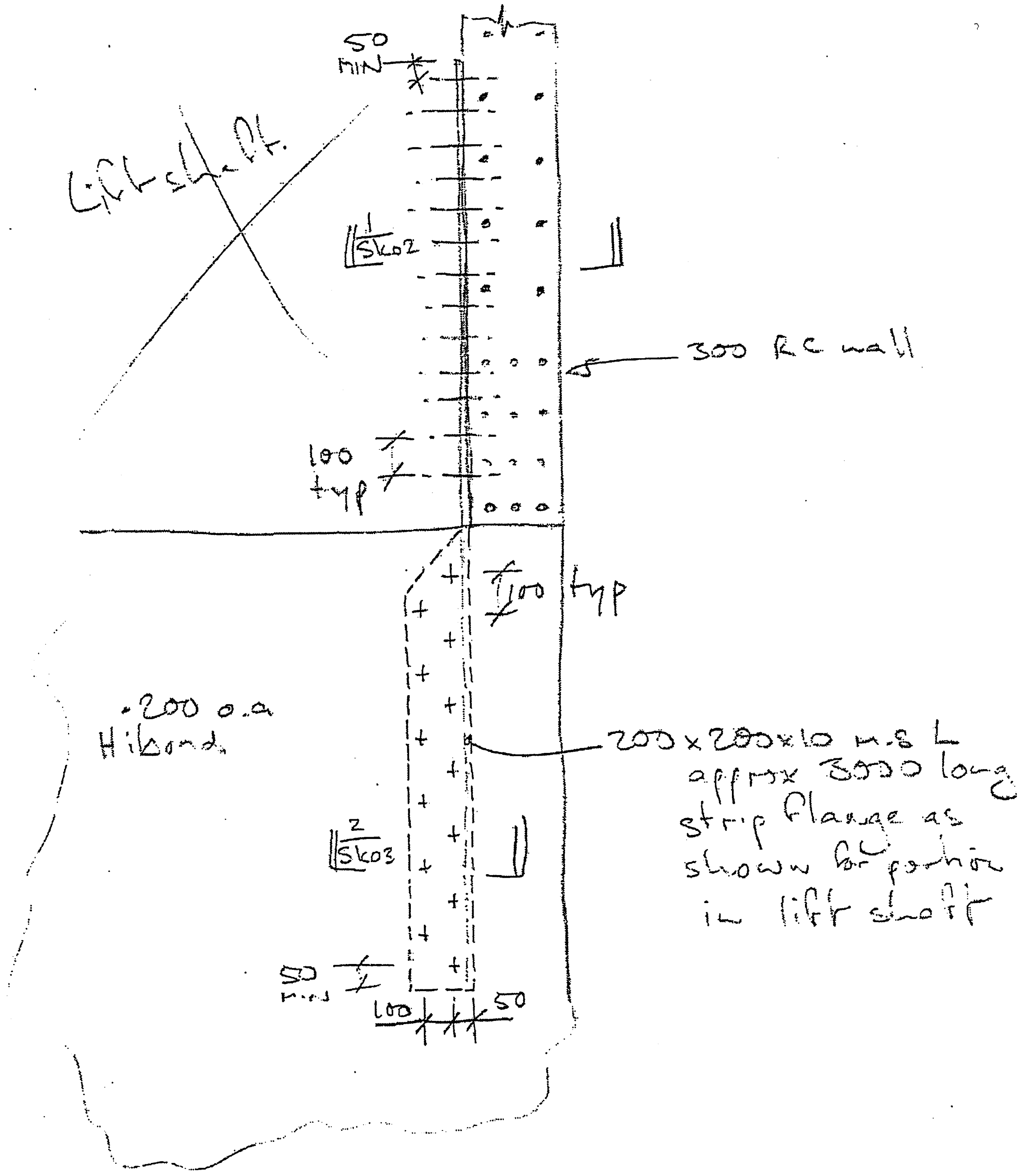
Holmes Consulting Group Limited,
61 Cambridge Terrace,
P.O. Box 701,
Christchurch, New Zealand.
Telephone: (03) 663-366.
Facsimile: (03) 792-169.

Offices in Christchurch, Wellington, New Plymouth, Auckland.

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

CALCULATIONS

JOB NAME
JOB No
CALCS BY
PAGE Sk-01
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.

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

CALCULATIONS

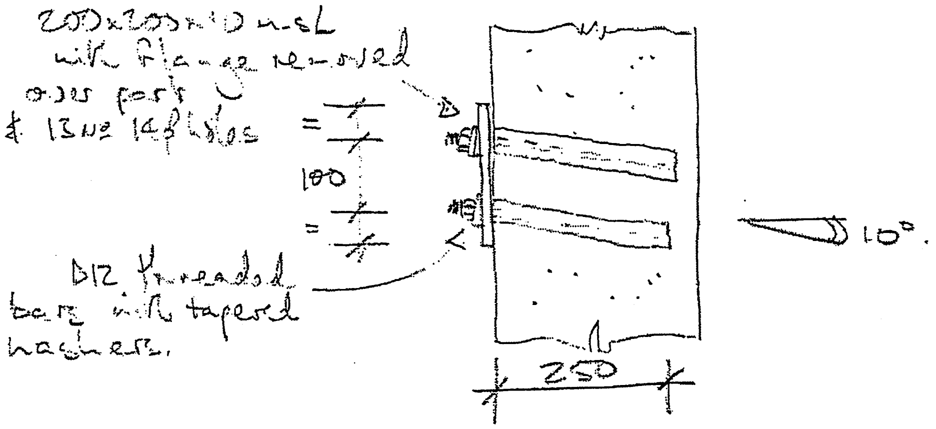
JOB NAME

PAGE SK-02

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

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

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 ~~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

COPIES TO

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

MEMO

JOB NAME 249 Madras St.
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.

COPIES TO

HOLMES CONSULTING GROUP
STRUCTURAL AND CIVIL ENGINEERS
Offices in Christchurch, Wellington, New Plymouth, Auckland.

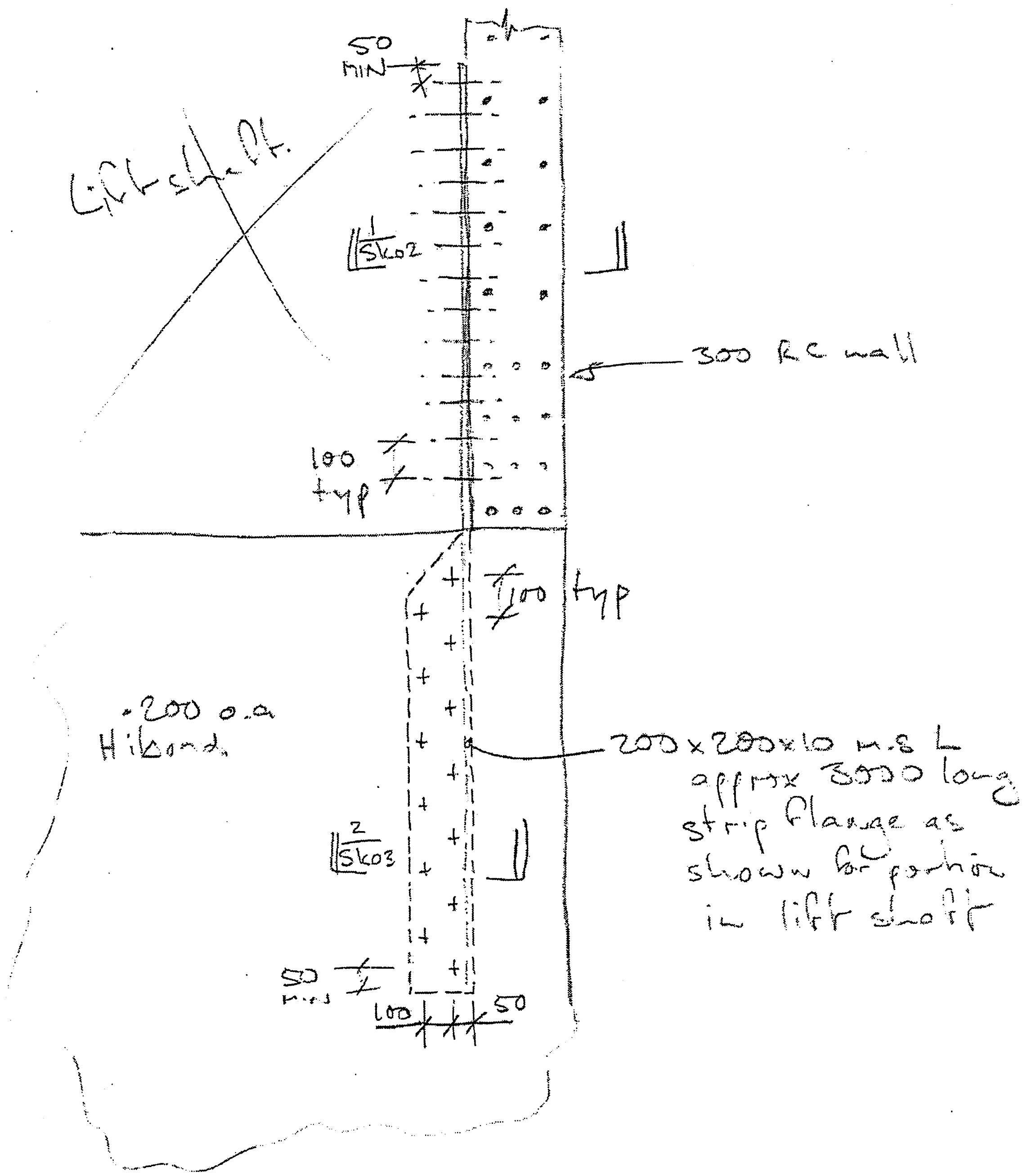
CALCULATIONS

JOB NAME

JOB No CALCS BY

PAGE SK-01

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.

HOLMES CONSULTING GROUP

STRUCTURAL AND CIVIL ENGINEERS

Offices in Christchurch, Wellington, New Plymouth, Auckland.

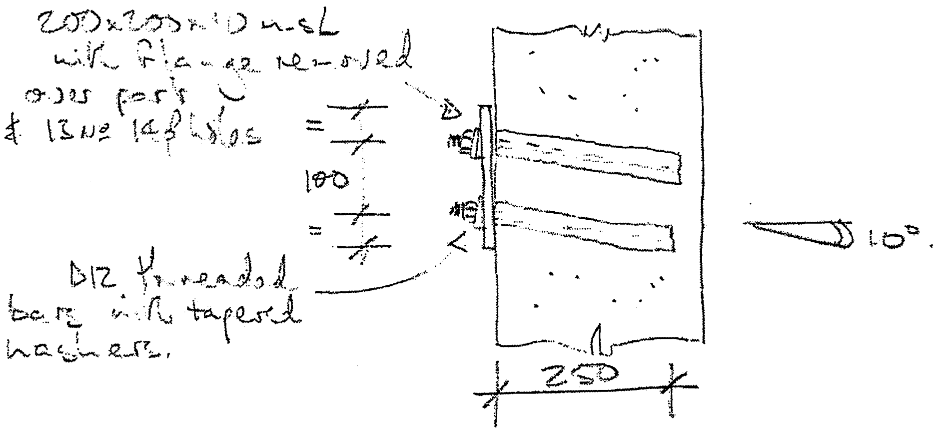
CALCULATIONS

JOB NAME

JOB No — CALCS BY

PAGE Sk-02

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

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

1 February 1990

Mr Keith Long
Claims Manager
Adam and Adam Limited
PO Box 2517
WELLINGTON

Dear Sir

RE: PROFESSIONAL INDEMNITY INSURANCE

We wish to confirm to you our notification of a potential claim on the above policy, as discussed with you by telephone today.

This situation relates to a five storey building at 249 Madras Street, built approximately three years ago. This firm was engaged by the builder on a design-build basis.

Both the builder, Williams Construction and the owner, Prime West Corporation, are now in receivership, and the owner's receiver is attempting to sell the property. A potential purchaser has an option until 28 February 1990 and has engaged Holmes Consulting Group to undertake a structural survey and present a report.

Holmes Consulting Group obtained some structural documents, and advised us that there appeared to be a deficiency in the detailing of the connection of several shear walls to the floor diaphragms. Our own review of the drawings confirms an apparent lack of ties to two walls.

We have contacted the engineer directly involved with the design and observation of the project (he is no longer employed by this practice). He was unable to recall any site instructions given on this item, and we have found no reference to it in the written instructions we have on file. We have forwarded to him a copy of the relevant drawings to help his recall, and are also attempting to contact the foreman involved.

We have used an electronic reinforcing bar locater at one level, which has indicated that some reinforcement is present, but not what quantity. The readings may also have been affected by metal work in the walls or the metal tray flooring system, and could not be totally relied on.

The receiver has today given us a copy of the report and advised that the purchaser's solicitor is requesting a two month delay in settlement to give time to do remedial work (a copy of the report is attached). We have estimated that the direct costs of remedial work, if required, would be in the region of $5000.00 to $10,000.00 but are aware that the indirect costs of a delayed or terminated sale agreement may be much higher. We estimate that the remedial work may take one or two weeks.

Our proposed course of action is as follows:-

a. To agree with Holmes Consulting on the precise scope of the work they consider may be inadequate.
b. To confirm with Holmes Consulting the level of load for which the floor to wall tie should be designed.
c. To design the remedial work that would be required if the ties are not present.

All of the above would be undertaken without admitting liability. Having spoken with Mr Peter Smith of CEAS, we understand that we have approval to proceed with items a. to c.

I trust that this is sufficient notice at present. Please contact myself or Dr Reay if you require any further information.

Yours faithfully

G.N. Banks

c.c. Mr Peter Smith
CEAS

ALAN REAY CONSULTANTS LIMITED

147 KILMORE STREET
BOX 25-026, VICTORIA ST
CHRISTCHURCH 1
Telephone: 880-434
Fax No: (03) 793-981

ALAN M. REAY
D.E.(Hons.), Ph.D.
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

KPMG Peat Marwick

Chartered Accountants

Office address
Clarendon Tower
78 Worcester Street
Christchurch
New Zealand

Mail address
P.O. Box 274
Christchurch
New Zealand

Telephone (03) 796-480
Fax No 0064 (03) 663101

2 February 1990

Mr Alan M Reay
Consulting Engineer
P O Box 25-028
CHRISTCHURCH

Dear Sir

RE: 249 MADRAS STREET AND PRIME WEST CORPORATION

Further to our meeting on 1 February 1990 with yourself and Mr Geoff Banks, we record our understanding of the steps to be taken with regard to the alleged non-compliance with current design codes as recorded in the structural report prepared by Holmes Consulting Group Limited, dated January 1990.

You have advised that investigations are continuing as to whether or not steel ties were placed between the structural floor and some shear walls as a metal detector has indicated the presence of some steel.

You have also advised that the cost of the remedial work would be approximately $5,000 and should take only one week's work to complete.

In view of the relatively modest cost for the remedial work, you have advised it is more cost effective to assume that the steel is not in place, as the cost of further investigating the matter would in all probability exceed this amount. You have also advised that there is reasonable agreement with Holmes Consulting Group as to the level of remedial work required, and that once carried out, there is no suggestion that the building is not at proper standard.

On an entirely without prejudice basis, you have offered to complete engineering drawings for the remedial works and presumably oversee their completion at your own cost. Both parties have reserved their positions with regard to who should bear the contractors' cost of carrying out the repairs.

To ensure that Holmes Consulting Group can promptly report to the Canterbury Regional Council that current design codes have been fully complied with, no doubt you will ensure that full agreement is obtained with them as to the level of the work required.

We have advised Mr Stock, Solicitor for the Canterbury Regional Council, that the remedial work is to be carried out forthwith and did not appear to be potentially expensive as intimated by Holmes Consulting Group. Further, we have advised that the work should take approximately a week to complete, and accordingly will not disrupt the Council's fit-out and move into the building.

We impressed upon you the extreme difficulty we have had locating a purchaser for this property and I am sure you appreciate that we must ensure that the sale is not put in jeopardy by restricting the Council's ability to take possession without delay, since it has been expressed to us that time is of the essence. Accordingly, we appreciate the prompt attention you have given to this matter and we would hope that a costing for these works could be arranged to enable commencement early next week.

Please advise if your understanding of the situation is not as set out above.

Yours faithfully
KPMG PEAT MARWICK

for P W Young

G\PWY\S\2.3

.nnity & General
INSURANCE COMPANY LIMITED

Adam
adam

COPY

12 . / 1990

Alan R..y Consultants Ltd
P O Box 25 028
CHRISTCHURCH

Attention Mr G Banks

Dear Geoff

I & G 990 ALAN REAY CONSULTANTS LTD -
CRIME WEST CORPORATION LTD

Further to our telephone discussion of the 1st February and receipt of your letter of same, we confirm our approval for you to agree with Holmes Consulting Ltd the precise scope of the work they consider to be inadequate, the level of load for which the floor to wall connection should be designed and the design of the remedial work.

The above should be undertaken on a without prejudice basis, with no admission of liability.

We understand that you will also pursue the possibility that adequate reinforcing was placed and that no remedial work may in fact be required.

The need for prompt action to prevent sequential loss is appreciated.

No commitment or financial undertaking should be entered into without the approval of claims committee and would you please keep claims committee fully informed of developments and include the I & G claim number I & G 990 on all correspondence relating to the claim.

Yours faithfully,

P C Smith

Claims Committee Member

From the Office of
P.C. Smith
Claims Director

P.O. Box 588, Wellington, Telephone (04) 722-261, Fax (04) 712-372

3808
14/02/90.

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

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

2. Confirmed the only system OK - possible pretension via nuts.

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

GB

WIT.HARE.0001.62
BUI.MAD249.0130.15

CALCULATIONS	249 MADRAS ST – DIAPHRAGM CHECK	PAGE	1
ALAN M. REAY CONSULTING ENGINEER CHRISTCHURCH		SECT	
		FILE	3608
		DATE	29/01/90

- Building has 5 suspended floor 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)
60% 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
$C_p = \alpha K_x Z R C_p$

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

$h_n = 20.50$

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

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

$Z = 5/6$ zone B

$R = 1.0$

$C_p = 0.3$

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

$\Rightarrow$ 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 dist^n 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) $\frac{741}{3298}(1241) = \underline{\underline{279 \text{ kN}}}$

CALCULATIONS

ALAN M. REAY
CONSULTING ENGINEER
CHRISTCHURCH

PAGE 3
SECT
FILE 3609
DATE 29/01/90

3 Design Connection - Reinforcement

3.1 Wall (4)

design V_u = 724 kN over 2.65m 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.

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

diaphragm

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

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

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

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

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 $A_{st} = \frac{231 \times 10^3}{0.9 \times 380} = 675 \text{ mm}^2$

CALCULATIONS

ALAN M. REAY
CONSULTING ENGINEER
CHRISTCHURCH

PAGE 4
SECT
FILE 3608
DATE 1/02/90

4. Design Connections - Bolted.

for walls 3+9, T_u = 279 kN max.

try fabricated angle:

bolts: (in shear) - use S_{es} = 3.3 for EQ

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

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

reqs.

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

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

$\Rightarrow$ N° reqd = $\frac{279}{38.7}$ = 7.2 ; say 7. (S_{es} 3.2)

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

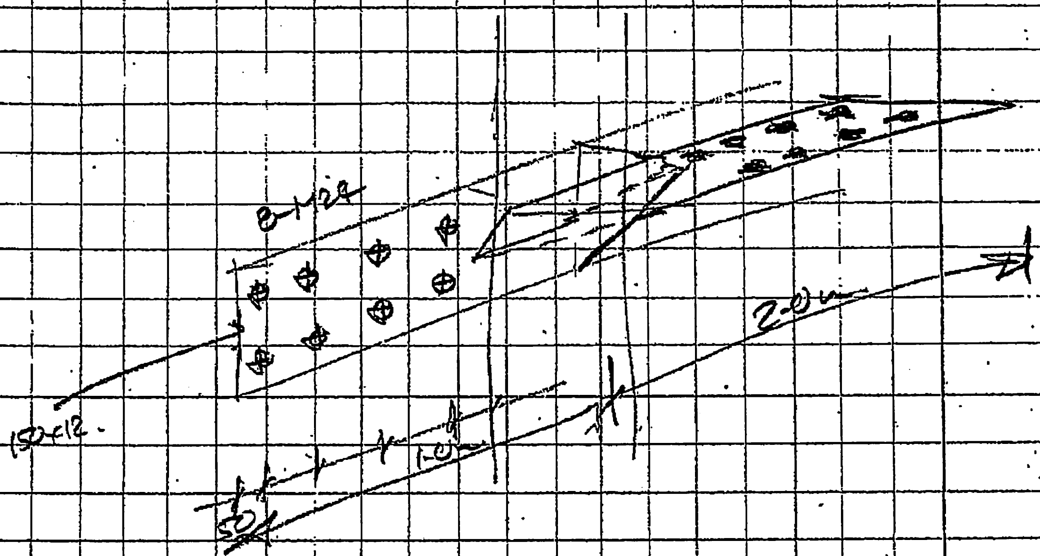
steel area : $A_t = \frac{279}{.250}$ = 1116 mm^2.

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

CALCULATIONS

ALAN M. REAY
CONSULTING ENGINEER
CHRISTCHURCH

PAGE 5
SECT
FILE
DATE



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

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

labour. $340.

$600 /unit.

CALCULATIONS

ALAN M. REAY
CONSULTING ENGINEER
CHRISTCHURCH

PAGE 6A
SECT
FILE 3608
DATE 10/10/91

5 Design Connection - Tie System.

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

5.1 Loads

		U_{des}
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

$$\text{allowable } f_c = (0.85 \times 0.70) 250 = 14.9 \text{ MPa}$$

⇒ compression easily OK.

CALCULATIONS

ALAN REAY CONSULTANTS LTD
Consulting Engineers
Christchurch

PAGE 7A.
SECT.
FILE 3608
DATE 10/10/91.

5.3 Tennyson Tce

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:

$$T_i = 0.9(\cdot113 \times 380)2 = 77 \text{ kN}.$$

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

				M20	M24 Chem.
level 5:	300 − 77	=	223 kN.	9	6
level 4:	240 − 77	=	163 kN	65 ~~7~~ 6	5
level 3:	184 − 77	=	107 kN.	4.	3

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

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

⇒ No bolts shown above

- recommended spacing = 160 mm.

⇒ detail bracket as shown over.

CALCULATIONS

ALAN REAY CONSULTANTS LTD
Consulting Engineers
Christchurch

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

• use H20 anchors to floor

$V_{H20} = \frac{85.7}{3.2} = 27$ kN. (see 7A for N's)

CALCULATIONS		PAGE	9A
ALAN REAY CONSULTANTS LTD Consulting Engineers Christchurch		SECT	
		FILE	3608
		DATE	10/10/91

5.3.2 Wall No 3

as wall 9 except only 1 Hilbar per level loaded.

⇒ design loads as follows:

			H20	H24
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: →
(Chamset)

6 Shear Transfer, Levels 1 and 2

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

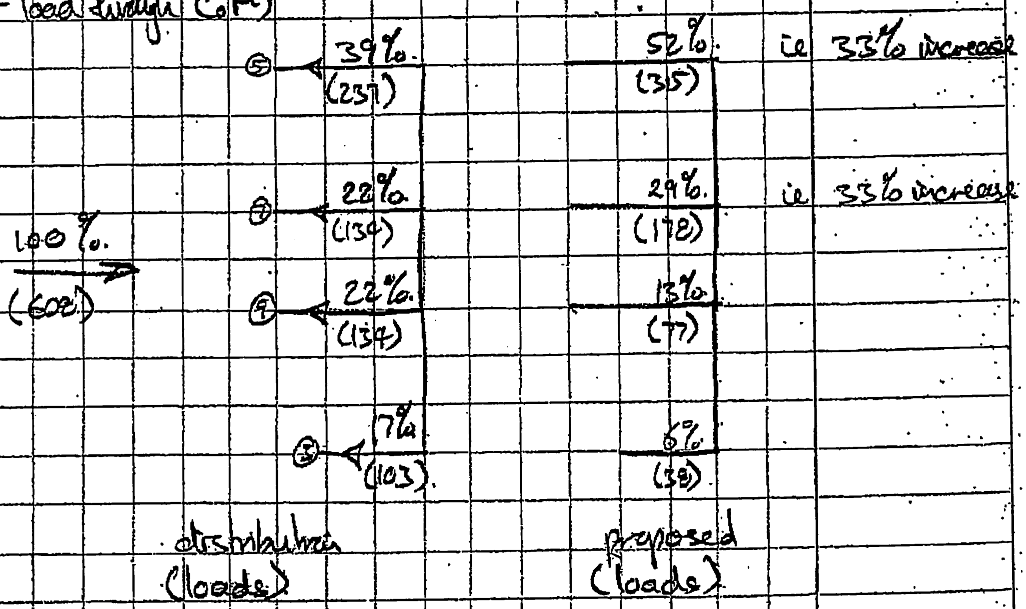
- design shears for diaphragm design are as follows:

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 C of M)



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

⇒ proposed revised distribution above.

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

Wall No 5 : ref. p347A of calcs

code advice : 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	2.57
	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

CALCULATIONS

ALAN REAY CONSULTANTS LTD
Consulting Engineers
Christchurch

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

Wall No 7 : · refer to p 348 of calcs

code actions :
· info not available on actions above ground floor

however :

flexure : max. possible moment increase is $(178-134)(6{\cdot}9+3{\cdot}7) = 466$ kNm.
on design Mode 3978 kNm, ie 12% incr.

but detail for ductility ⇒ ok.

(likely bar strength margin = $\frac{430}{380} = 1{\cdot}13$) ⇒ ok

shear : governed by overstrength
– details as wall 7, but design shear less ⇒ ok.

flexure & shear OK