DISCIPLINARY COMMITTEE DECISION

COMPLAINT ABOUT KEVIN O'CONNOR

For release

In accordance with:

Chartered Professional Engineers of New Zealand Act 2002

Chartered Professional Engineers of New Zealand Rules (No 2) 2002

Institution of Professional Engineers New Zealand Rules

Institution of Professional Engineers New Zealand Disciplinary Regulations

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Hamish Wilson, nominated by Consumer New Zealand

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engineering new zealand

Institute of Engineering Professionals

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EXECUTIVE SUMMARY

1. In 2015, Engineering New Zealand[1] was made aware of concerns surrounding the structural design of six buildings in Masterton owned by Masterton Trust Lands Trust (MTLT). The concerns were raised by a Chartered Professional Engineer (CPEng). The matter was referred to the Ministry of Business, Innovation and Employment (MBIE), as the relevant regulatory authority.

2. Following the publication of two reports commissioned by MBIE in which concerns about the structural integrity of the six buildings were raised, Engineering New Zealand initiated an own-motion inquiry into their design. Mr O'Connor was identified as the professional engineer who had signed the producer statement for the design of five of the buildings investigated.

DECISION

3. Having considered the reasons given by the Investigating Committee, and the parties' agreed statement of facts, the Disciplinary Committee finds that the engineering services provided by Mr O'Connor in signing the producer statements, and thereby taking responsibility for designs that were subsequently found to be inadequate, did not meet the standard to be reasonably expected from a Chartered Professional Engineer and a member of Engineering New Zealand.

4. Accordingly, the Disciplinary Committee has upheld the complaint.

[1] On 1 October 2017 the Institution of Professional Engineers New Zealand changed its trading name from IPENZ to Engineering New Zealand.

BACKGROUND

COMPLAINT

5. In 2015, Engineering New Zealand was made aware of concerns surrounding six buildings in Masterton owned by Masterton Trust Lands Trust (MTLT). The concerns were raised by a Chartered Professional Engineer (CPEng).

6. The concerns were about the structural integrity of the buildings. Engineering New Zealand, having no jurisdiction over physical assets, brought the concerns to the attention of the Ministry of Business, Innovation and Employment (MBIE), as the relevant regulatory authority.

7. MBIE subsequently commissioned GA Hughes & Associates Ltd to carry out a structural review of the buildings,[2] which identified concerns with the buildings and recommended a Detailed Seismic Assessment be carried out. Following the receipt of this report, MTLT and MBIE commissioned a Detailed Seismic Assessment to be carried out by Holmes Consulting who identified concerns about the structural integrity of the six buildings. MTLT subsequently commenced remedial works on some of the buildings.

8. In light of the findings of these two reports, Engineering New Zealand decided that it needed to act on this information to determine if there is an issue with the engineering design of these buildings or not and, if so, whether there were grounds for discipline against the engineer or engineers responsible. It commenced an own-motion inquiry pursuant to Rule 55 (1) of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002 (the Rules).

9. The scope of the inquiry was to investigate:

   "The circumstances relating to the design, design review and construction monitoring of six buildings in Masterton:

   - to assess whether the Chartered Professional Engineers involved have provided engineering services in accordance with accepted standards; and
   - to learn and advise on any engineering performance and practice improvements, if necessary."

10. Kevin O'Connor & Associates Ltd (KOA Ltd) was the engineering firm involved in the engineering design of all six of the buildings. Kevin O'Connor CPEng[3], CMEngNZ[4], IntPE(NZ)[5] was identified as the Chartered Professional Engineer who signed the Producer Statement – Design (PS1) for four of the buildings:

    - 57-65 Dixon Street, designed and built in 2006;
    - 57-65 Dixon Street, designed and built in 2010/2011;
    - 96-120 Queen Street, designed and built in 2011; and
    - Corner of Dixon and Church Streets FMG Building, designed and built in 2014.

[2] One of the buildings that Engineering New Zealand was initially notified about was different from the six ultimately chosen by MBIE for further assessment.

[3] Chartered Professional Engineer.

[4] Engineering New Zealand Chartered Member.

[5] A member of the New Zealand section of the International Professional Engineers.

While he was an Engineering New Zealand member, but not a CPEng, he also signed the PS1 for:

- 408 Queen Street, designed and built in 2003.

INVESTIGATING COMMITTEE'S DECISIONS

11. Following an initial investigation this matter was referred to an investigating committee for formal investigation.

12. The Investigating Committee considered whether Mr O'Connor provided engineering services relating to the engineering design work on the five buildings identified above in accordance with accepted standards.

13. The Investigating Committee obtained independent expert advice from Barry Brown FEngNZ[6] CPEng IntPE (NZ) and Stuart George CMEngNZ CPEng IntPE(NZ).

14. The Investigating Committee issued a separate decision document for each building.

15. The Investigating Committee considered that it was inappropriate for Mr O'Connor to sign the PS1s for the five buildings. It was their opinion that Mr O'Connor should have identified the issues noted by the Committee's independent experts when reviewing the designs for sign-off and taken additional steps to reassure himself the designs met the relevant standards. They considered this was more than a minor departure from accepted standards.

16. Accordingly, the Investigating Committee did not consider that there were grounds to reasonably dismiss the matter based on the information available to them and decided to refer it to a disciplinary committee in accordance with rule 60(a) of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002 and with clause 11(a) of the IPENZ Disciplinary Regulations in the case of the building at 408 Queen Street.

DISCIPLINARY COMMITTEE

17. The members of the Disciplinary Committee are:

    Jenny Culliford FEngNZ (Ret.) (Chair)
    Peter Boardman FEngNZ CPEng IntPE(NZ)
    Don Thomson CMEngNZ CPEng IntPE(NZ)
    Hamish Wilson, nominated by Consumer New Zealand
    Anita Killeen, Barrister and Solicitor of the High Court of New Zealand

Pre-hearing matters

18. On 13 October 2020, counsel for Mr O'Connor wrote to the Chair of the Disciplinary Committee formally requesting a temporary stay of the disciplinary process pending determination of related Court claims in which KOA Ltd and Mr O'Connor are defendants. Their professional indemnity insurers are conducting the defence. Counsel submitted that:

[6] Fellow of Engineering New Zealand.

a. The insurers' contractual right to conduct the defence could be compromised or undermined by the Disciplinary Committee's findings if they are against Mr O'Connor "with inevitable publication of such a result"; and

b. Mr O'Connor's entitlement to a fair hearing of defences advanced in the Courts could be compromised by any early and adverse decision in the disciplinary process.

19. Counsel for the Registration Authority (RA) responded on 10 November 2020 submitting that an adjournment was not appropriate for the following reasons:

a. "any finding by the Disciplinary Committee would not prejudice Mr O'Connor's right to a fair trial as different standards apply to negligence in a professional disciplinary context and tortious negligence;

b. an adjournment would cause significant delay which is likely to prejudice the Disciplinary Committee's ability to fairly hear the RA's evidence; and

c. the prejudice to the RA arising from the adjournment outweighs the prejudice to Mr O'Connor. "

20. On 13 November 2020, the Chair of the Disciplinary Committee wrote to counsel for Mr O'Connor declining the request for a temporary stay of the disciplinary process.

21. On 1 December 2020, counsel for the parties filed an agreed statement of facts (dated 19 November 2020) stating that:

a. "Mr O'Connor accepts that the designs of the Masterton Buildings were inadequate and not in accordance with the standards reasonably expected of a Chartered Professional Engineer (or, in the case of the building at 408 Queen Street, of a member of Engineering New Zealand)."

b. "Mr O'Connor accepts that he had a responsibility to be satisfied on reasonable grounds that the designs were adequate and in accordance with the requisite standards of the time before signing the PS1."

c. "Mr O'Connor further admits that the admitted facts establish grounds for discipline under section 21 of the Act in that Mr O'Connor has performed engineering services in a negligent manner (s21(1)(c)) and that there are grounds for ordering a disciplinary penalty under section 22 of the Act and, in the case of the building at 408 Queen Street, under rule 11 of the Engineering New Zealand Rules and reg 45 of the Engineering New Zealand Disciplinary Regulation."

22. On 3 February 2021, the Chair of the Disciplinary Committee wrote to the parties advising that the Committee agreed that there were grounds for discipline and proposing to dispense with a hearing. The parties were invited, should they have no objection to the process proposed by the Committee, to make submissions on the appropriate disciplinary sanction, allocation of costs and publication. Annexed is a copy of the agreed statement of facts.

DISCUSSION

THE DISCIPLINARY COMMITTEE'S ROLE

23. Professional disciplinary processes primarily exist to protect the public, uphold professional standards, and maintain public confidence in the profession and its regulation. They do this by ensuring that members of the profession adhere to certain universal (or accepted) professional standards.[7]

24. The role of the Disciplinary Committee in the disciplinary process is to consider whether Mr O'Connor has acted in accordance with accepted professional standards and, if not, whether there are grounds for disciplining him in accordance with the Chartered Professional Engineers of New Zealand Act 2002 and the IPENZ Rules and Disciplinary Regulations.[8]

THE LEGAL TEST

25. The legal test to assess whether Mr O'Connor acted in accordance with acceptable professional standards is whether he acted in accordance with what a reasonable body of his peers would have done in the same situation.

26. The assessment of whether an engineer has acted in accordance with accepted standards may be informed by whether reasonable members of the public would "consider such an act or omission, if acceptable to the profession, were to lower the standard of that profession in the eyes of the public".[9]

27. If the evidence is that Mr O'Connor acted in accordance with accepted standards, then we will dismiss the complaint. If the evidence is that Mr O'Connor did not act in accordance with accepted standards, then we will uphold the complaint. If the behaviour meets the latter criterion, we must consider whether the conduct "falls seriously short of accepted conduct" before imposing a disciplinary sanction.[10]

28. This means that the matter for the Disciplinary Committee to decide in this case is whether the engineering services provided by Mr O'Connor, as agreed to by the parties, met the standard to be reasonably expected of a Chartered Professional Engineer and a member of Engineering New Zealand.

29. Our approach to this question has been to consider the agreed statement of facts, and the analysis and findings of the Investigating Committee and the information that formed the basis of their decisions.

ANALYSIS

30. We have considered the Investigating Committee's decisions and the parties' agreed statement of facts for each of the five buildings covered by the complaint.

31. In each case Mr O'Connor has signed the PS1 for a building that has upon later review by third parties been found to have issues with the adequacy of the structural design.

[7] *Dentice v Valuers Registration Board* [1992] 1 NZLR 720 (HC).

[8] When referring to the Rules or Disciplinary Regulations, we refer to IPENZ Rules and the accompanying Disciplinary Regulations that were in place at the relevant time.

[9] *Robinson v RA* (10 July 2015, *Appeal Ruling* #29) Chartered Professional Engineers Council. Available at: **http://www.cpec.org.nz/appeal-rulings/appeal-21-10-july-2015-robinson-v-ra**.

[10] Ibid.

32. By signing the PS1s, Mr O'Connor was confirming his professional opinion that aspects of the buildings' designs complied with the Building Code. As noted by the Investigating Committee, a PS1 signals to a building consent authority (BCA) that certain design work has been done (or overseen/supervised) by a practitioner who is competent to perform the defined work. Whilst producer statements have no legal status under the Building Act, they are commonly relied upon by BCAs when determining whether there are reasonable grounds to conclude that engineering work complies with the Building Code.

33. Mr O'Connor has admitted that his reviews/checks prior to signing the PS1s have been high-level and that he has generally relied on the reviews carried out by a senior engineer. He did not know the extent of the checks that had been carried out and would take the engineer's word that the check had taken place. There is no evidence that the checking process was documented.

34. We are concerned that there is a pattern of behaviour over a sustained period of time where Mr O'Connor has signed off, and taken responsibility for, a building design by signing a PS1 with limited information to justify the sign off – generally his own cursory review of the design and reliance on the checking engineer's word that checks had been undertaken.

35. Mr O'Connor has accepted that the designs of the Masterton buildings were inadequate and not in accordance with the standards reasonably expected of a chartered professional engineer (or, in the case of the building at 408 Queen Street, of a member of Engineering New Zealand).

36. Furthermore, Mr O'Connor has acknowledged that he had a responsibility to be satisfied on reasonable grounds that the designs were adequate and in accordance with the requisite standards of the time before signing a PS1.

GROUNDS OF DISCIPLINE

37. The Disciplinary Committee may make an order for discipline against a Chartered Professional Engineer and a Chartered Member of Engineering New Zealand if it is satisfied that the engineer concerned has performed engineering services in a negligent or incompetent manner.

38. To determine whether Mr O'Connor acted negligently or incompetently we refer to the decision of the Chartered Professional Engineers Council in *R v K*:[11]

> *The starting point is to consider what standard sets the benchmark for negligent or incompetent behaviour. We consider that incompetence is a more serious allegation than negligence. One can be negligent without being incompetent, but it is highly unlikely that someone who is incompetent is not also negligent.*

39. Further, *Robinson v RA* states:[12]

> *Whether engineering services have been performed in an incompetent manner is a question of whether there has been a serious lack of competence (or deficit in the required skills) judged by the areas of competence which in this case are encapsulated by Rule 6* [of the Chartered Professional Engineers Rules (No 2) 2002 (the Rules)].

[11] *R v K,* Appeal Ruling 11/14, Chartered Professional Engineers Council at [36] and [38].
[12] *Robinson v RA* (10 July 2015, *Appeal Ruling* #29) Chartered Professional Engineers Council at [40(c)].

40. Chartered Professional Engineers are assessed against the 12 elements set out Rule 6 of the Rules to establish their competence, they are:

(a) comprehend, and apply his or her knowledge of, accepted principles underpinning—

(i) widely applied good practice for professional engineering; and

(ii) good practice for professional engineering that is specific to New Zealand; and

(b) define, investigate, and analyse complex engineering problems in accordance with good practice for professional engineering; and

(c) design or develop solutions to complex engineering problems in accordance with good practice for professional engineering; and

(d) exercise sound professional engineering judgement; and

(e) be responsible for making decisions on part or all of 1 or more complex engineering activities; and

(f) manage part or all of 1 or more complex engineering activities in accordance with good engineering management practice; and

(g) identify, assess, and manage engineering risk; and

(h) conduct his or her professional engineering activities to an ethical standard at least equivalent to the code of ethical conduct; and

(i) recognise the reasonably foreseeable social, cultural, and environmental effects of professional engineering activities generally; and

(j) communicate clearly to other engineers and others that he or she is likely to deal with in the course of his or her professional engineering activities; and

(k) maintain the currency of his or her professional engineering knowledge and skills.

41. We agree with the Investigating Committee that it was inappropriate for Mr O'Connor to sign the PS1s. As they note in their conclusions, Mr O'Connor should have identified the design issues found by the Committee's experts when he reviewed the designs for sign off. He should also have taken additional steps to assure himself that the designs met the relevant standards.

42. We concur with the Investigating Committee's finding that there was no requirement for Mr O'Connor to undertake "exhaustive" reviews of the designs, but as the person signing the PS1 he had a responsibility to carry out an adequate review himself rather than rely on the reviews that may, or may not, have been carried out by designers and checkers during the design process. We agree with the Investigating Committee's view that a reasonable engineer would document any process of checking and clarifying aspects of a design at the sign off stage.

43. We also agree with the Investigating Committee that a reasonable body of Mr O'Connor's peers would likely consider that his actions in signing the PS1s without adequate design review was not consistent with accepted practice at the time.

44. In our view, Mr O'Connor has been negligent not incompetent. There is no evidence that he lacks competence. He is a very experienced structural engineer whose company operated successfully until the investigations into the Masterton buildings with the associated publicity occurred.

45. We therefore conclude that Mr O’Connor has met the grounds for discipline under section 21(1)(c) of the Chartered Professional Engineers of New Zealand Act 2002 and regulation 17 of the IPENZ Disciplinary Regulations.

DECISION

46. Having considered all the evidence, and the parties’ agreed statement of facts, we have decided to uphold the complaint about Mr O’Connor. We find that Mr O’Connor has performed engineering services in a negligent manner.

47. Having found Mr O’Connor in breach of his obligations as a chartered professional engineer and a member of Engineering New Zealand, we need to determine what orders, if any, should be made against him.

ORDERS

48. There is a range of disciplinary actions available to the us as set out in section 22(1) of the Act. There is also a range of sanctions in respect of Mr O’Connor’s membership with Engineering New Zealand under IPENZ Disciplinary Regulation 17(3).

49. Submissions on penalty were received from Engineering New Zealand on 23 February 2021 and from Mr O’Connor on 3 March 2021. Supplementary submissions on costs were received from Engineering New Zealand on 17 March 2021 and from Mr O’Connor on 18 March 2021. Submissions as to a fine were received from Engineering New Zealand on 30 March 2021 and from Mr O’Connor on 31 March 2021.

RELEVANT LAW

50. In *Roberts v A Professional Conduct Committee of the Nursing Council of New Zealand*[13] the High Court outlined a number of principles to be applied by the Health Practitioners Disciplinary Tribunal in determining the appropriate penalty to impose in disciplinary proceedings. The High Court determined that a disciplinary penalty must:

   a. protect the public (including through deterrence of other practitioners from engaging in similar conduct);
   b. set and maintain professional standards;
   c. where appropriate, rehabilitate the practitioner back to the profession;
   d. be comparable with penalties imposed on practitioners in similar circumstances;
   e. reflect the seriousness of the practitioner’s conduct, in light of the range of penalties available;
   f. be the least restrictive penalty that can reasonably be imposed in the circumstances; and
   g. be fair, reasonable, and proportionate in the circumstances.

51. The High Court also stated that while penalty may have the effect of punishing a practitioner, punishment is not a necessary focus for the Tribunal in determining penalty.

[13] [2012] NZHC 3354.

52. The principles in *Roberts* are broadly applicable to our power to make disciplinary orders under section 22 of the Act and IPENZ Disciplinary Regulation 17 and they are the principles we rely on when considering the appropriate penalty orders in this case.

53. The principles have general application to professional disciplinary proceedings in the light of the Supreme Court's decision in *Z v Dental Complaints Assessment Committee*.[14] In *Z,* the Supreme Court makes general statements about the purposes of professional disciplinary proceedings, noting that such proceedings are designed to:

> *Ascertain whether a practitioner has met appropriate standards of conduct in the occupation concerned and what may be required to ensure that, in the public interest, such standards are met in the future. The protection of the public is the central focus.*

54. This is consistent with *Roberts*, as *Roberts* lists public protection and the maintenance of professional standards as the foremost considerations relevant to penalty.

55. The Supreme Court in *Z v Dental Complaints Assessment Committee*[15] also states that while professional disciplinary proceedings are not intended to punish practitioners, they may have a punitive effect in practice. This is also consistent with the principles set out in *Roberts*, in that the penalty must be the least restrictive penalty and that punishment is not a necessary focus of a disciplinary penalty.

56. The reasoning underlying *Roberts'* focus on practitioner rehabilitation is less relevant to penalties under the Act in light of the fact that the removal or suspension of a Chartered Professional Engineer's registration does not prevent the individual practising as an engineer but does prevent use of the Chartered Professional Engineer title.

57. It is appropriate that disciplinary penalties mark the profession's condemnation of the relevant conduct, noting that to do otherwise would not be consistent with the purpose of the Act to establish the title of Chartered Professional Engineer as a mark of quality.[16]

ENGINEERING NEW ZEALAND'S SUBMISSIONS

58. Engineering New Zealand submitted that Mr O'Connor:

    a. be censured by the Disciplinary Committee;

    b. be suspended for a period of 12 months or until he has fulfilled requirements for professional development as have been specified by the Disciplinary Committee; and

    c. the Disciplinary Committee's decision be published on the Engineering New Zealand website.

59. Engineering New Zealand submitted that there were several aggravating factors to be taken into account:

    a. Mr O'Connor's knowledge of his obligations as an engineer who qualified in 1983 and a CPEng since 2004;

[14] [2008] NZSC 55.
[15] Ibid.
[16] Chartered Professional Engineers of New Zealand Act 2002, s 3.

b. the number of buildings involved;

c. the significance of concerns with structural integrity: and

d. publicity associated with the concerns raised and investigations that followed.

60. Mitigating factors noted by Engineering New Zealand were:

a. Mr O'Connor's cooperation and engagement with the investigation process; and

b. Mr O'Connor's acceptance of responsibility for his actions.

61. Regarding publication and naming, Engineering New Zealand submitted that there were no circumstances in this case that would displace the presumption that Mr O'Connor would be named.

> *While we accept that Mr O'Connor is in the midst of civil proceedings relating to these buildings, we do not consider that this warrants name suppression. The claim against Mr O'Connor by Masterton Lands Trusts is one of tortious negligence which is a different standard and inquiry than that which is applied in professional negligence cases. It does not follow from a finding that an engineer has been professionally negligent that they are also liable for negligence in the tortious sense.*

62. Engineering New Zealand sought an order that Mr O'Connor pays 40% of the costs incurred by Engineering New Zealand amounting to $88,838.00. They have taken a starting point of 50% and reduced this to 40% to take account of Mr O'Connor's cooperation and recognising the significant admissions that he has made.

63. In response to the Disciplinary Committee's request for further information on the quantum of costs incurred, Engineering New Zealand confirmed total costs were $220,845 comprising internal staff costs of $22,100, investigator costs of $105,000, external legal fees amounting to $70,000, Disciplinary Committee costs of $6,900 and expert advice costs of $16,845.

64. Engineering New Zealand submitted that the investigation was one of the most complex it has undertaken, "involving multiple buildings and complicated structural engineering issues requiring expert advice". Due to the amount of documentation and complexity of the investigation, Engineering New Zealand contracted an investigator to conduct the initial investigation and prepare the matter for adjudication. Engineering New Zealand submit that they have taken a conservative approach to estimating the time spent on the inquiry internally and by their investigator and the applicable hourly rates.

65. Engineering New Zealand did not seek a fine in their first submissions but requested an opportunity to make submissions on an appropriate fine should the Committee decide to impose one. In response to a request from the Disciplinary Committee, they submitted that a fine of $2,000 would be appropriate.

MR O'CONNOR'S SUBMISSIONS

66. Mr O'Connor submitted that he accepted that censure would be inevitable and that a period of suspension with a requirement for professional development would be consistent with other disciplinary outcomes.

67. Mr O'Connor submitted that his name should not be published. He states that "His business has folded, he has not found suitable employment, he has been de facto suspended since March 2020, and he is still dealing with related claims in the Courts".

68. In his request in October 2020 for a temporary stay of the disciplinary process pending determination of related Court claims in which KOA Ltd and Mr O'Connor are defendants, Mr O'Connor was concerned that an adverse finding may prejudice his defence and his entitlement to a fair hearing. He was also concerned that his insurers' contractual right to conduct the defence could be compromised or undermined by publication of a disciplinary finding against him.

69. Mitigating factors submitted by Mr O'Connor include:

    a. the buildings have not failed;

    b. whilst accepting that by his signature on the PS1s he assumed responsibility for the designs, it was the same error repeated: "he knew of the professional qualifications and experience of the employed design engineers, he knew that they were not working outside their practice areas, the structures were relatively simple, and he had confidence in them reinforced by high level reviews and one to one interrogation"; and

    c. "with the rapid growth of KOA and his attendance at offices in Wellington and Auckland he was distracted and did not give the task the proper attention to detail that he should have".

70. Mr O'Connor submitted that both the costs incurred and the order sought by Engineering New Zealand are excessive when compared with costs disclosed in published decisions. He submitted that an order for $88,300 would be unduly harsh, disproportionate, and punitive. He further submitted that an order anywhere above $25,000 would be unduly punitive and out of parity with other disclosed disciplinary outcomes.

71. Counsel for Mr O'Connor seeks leniency having regard to mitigating factors and proportionality. He states that Mr O'Connor is insolvent with personal liabilities far exceeding his assets.

72. In response to Engineering New Zealand's supplementary submissions on costs, Mr O'Connor reiterated his position that costs incurred and sought are excessive and disproportionate.

    Looked at in the round, ENZ initiated an inquiry into a series of buildings in response to concerns expressed by MBIE and doubtless to assure MBIE that ENZ could and would investigate even though the buildings had not failed. It is not clear whether the costs referred to are limited only to the buildings relevant to the hearing before this committee involving Mr O'Connor, or they include costs relevant to buildings which were not the subject of ongoing disciplinary process and/or involved other engineers if they were. The total as claimed is not accepted as fair or reasonable costs incurred dealing with the matters that have been brought to this committee involving Mr O'Connor.

73. Mr O'Connor concurred with Engineering New Zealand's supplementary submission regarding an appropriate fine.

DISCUSSION

74. The public places significant trust in engineers to self-regulate. As a professional, an engineer must take responsibility for being competent, performing engineering activities in a careful manner and acting ethically. The actions of an individual engineer also play an important role in the way in which the profession is viewed by the public.

75. In our view, the respondent's actions, if condoned, would undermine the public's trust in the engineering profession and reduce the public confidence in the Chartered Professional Engineer title

and membership with Engineering New Zealand. The respondent's actions showed a lack of judgement and a lack of care. Whilst one such departure from the accepted standards might be considered to be towards the lower end of the scale, we view multiple breaches over several years as being very serious, and our orders need to reflect this view.

76. The Disciplinary Committee has found that Mr O'Connor has departed from what could be expected of a reasonable engineer. That is, Mr O'Connor has acted negligently.

Registration and membership

77. In respect of orders relating to registration as a Chartered Professional Engineer, the Disciplinary Committee may order that: [17]

- an engineer's registration be removed, and that they may not apply for re-registration before the expiry of a specified period;
- that their registration be suspended for a period of no more than 12 months or until they meet specified conditions relating to the registration; or
- that the engineer be censured.

Only one of these orders may be made in relation to a case.

78. In respect of orders relating to membership with Engineering New Zealand, the Disciplinary Committee may order that an Engineering New Zealand member be:[18]

- expelled from membership;
- suspended from membership for any period;
- suspended from membership until such time as the member has fulfilled requirements for professional development as have been specified by the Committee;
- suspended from membership for a period of time if by a prescribed date, the member fails to fulfil requirements for professional development as has been specified by the Committee.

79. In *A v Professional Conduct Committee*[19] the High Court said, in relation to a decision to cancel or suspend a professional's registration, that four points could be expressly and a fifth impliedly derived from the authorities:

> *First, the primary purpose of cancelling or suspending registration is to protect the public, but that 'inevitably imports some punitive element.' Secondly, to cancel is more punitive than to suspend and the choice between the two turns on what is proportionate. Thirdly, to suspend implies the conclusion that cancellation would have been disproportionate. Fourthly, suspension is most apt where there is 'some condition affecting the practitioner's fitness to practise which may or may not be amendable to cure'. Fifthly, and perhaps only implicitly, suspension ought not to be imposed simply to punish.*

[17] Chartered Professional Engineers of New Zealand Act 2002, s 22.
[18] IPENZ Disciplinary Regulations, reg 17(3)(a) – (d).
[19] *A v Professional Conduct Committee* [2008] NZHC 1387 at [81].

80. In the recent decision of *Attorney-General v Institution of Professional Engineers New Zealand Incorporated and Reay*[20] the High Court set out the standard the public expects when an engineer is a member of Engineering New Zealand:

> *[M]embership of a professional body, such as the Institution, can confer a status that signals trustworthiness to the public. This status reflects the value that society places upon the training and skill acquired by members and upon the Institution's ability to maintain the standards of its members through ongoing education, training and disciplinary processes.*

81. The Court also went on to set out the public expectation of Engineering New Zealand's role in maintaining the standard of the profession:[21]

> *There is, however, a counterbalance to the public trust that is reposed in members of professional bodies such as the Institution. That counterbalance is the public expectation that the Institution will tightly regulate admission into its ranks and ensure members maintain high professional standards. The public expects that if a person is to be afforded the status of membership of the Institution, then those individuals will maintain professional standards and that those standards will be enforced by the Institution through, if necessary, disciplinary proceedings. If a professional body, such as the Institution, wishes to maintain that public trust, and the value associated with membership status, then it must act in accordance with this expectation.*

82. We consider that the Court's comments in respect of membership with Engineering New Zealand are equally applicable to its role as the Registration Authority and regulator of Chartered Professional Engineers.

83. We gave careful consideration as to whether Mr O'Connor's registration as a Chartered Professional Engineer or Chartered Member of Engineering New Zealand should be removed or suspended. Engineering New Zealand sought suspension of his CPEng registration with orders for a period of professional development and Mr O'Connor accepted that this was a likely outcome.

84. We have been guided by the High Court's comments in *A v Professional Conduct Committee* (see above). The primary purpose of cancelling or suspending registration is protection of the public. We do not consider that Mr O'Connor's practice poses a risk to the public such that we would need to remove or suspend him. Furthermore, we do not consider that there would be benefit in suspending Mr O'Connor and requiring him to undertake professional development before reinstatement. We have found Mr O'Connor to be negligent, not lacking competence. The last building design signed off by Mr O'Connor covered by this complaint was completed in 2014.

85. Mr O'Connor has accepted responsibility for his actions and acknowledged that his conduct was not in accordance with standards he should have met. Mr O'Connor will undoubtedly have learned many lessons from the investigations, reviews, and inquiry into his work and that of his firm since 2016. If the circumstances were different, a period of suspension with professional development requirements may well have been appropriate.

[20] [2018] NZHC 3211 at [52] and [55].
[21] Ibid at [56].

86. Accordingly, we have decided that the appropriate penalty is censure and a fine.

Fine

87. The Chartered Professional Engineers of New Zealand Act 2002 and the IPENZ Disciplinary Regulations state that the Disciplinary Committee may order that an engineer pay a fine up to a maximum of $5,000.

88. Engineering New Zealand submitted that a fine of $2,000 would be appropriate and the respondent agreed.

89. We, however, consider that a larger fine is warranted taking into account the seriousness of the conduct, the number of buildings involved and the time over which the conduct occurred. This was not a one-off situation. We have set the fine at $3,500, a sum we consider to be proportionate whilst taking account of recent orders.

90. As stated above, Mr O'Connor's behaviour fell below the standard expected of a professional engineer, and it is important that the Registration Authority and Engineering New Zealand condemns this behaviour, and that this condemnation is reflected in the penalty ordered.

Costs

91. The Disciplinary Committee can order that the engineer pay costs and expenses of, and incidental to, the inquiry by Engineering New Zealand and Registration Authority.[22] We note the ordering of payment of costs is not in the nature of penalty.

92. When ordering costs, it is generally accepted that the normal approach is to start with a 50% contribution.[23] That, however, is a starting point and other factors may be considered to reduce or mitigate that portion. Those factors include whether the hearing was able to proceed on an agreed statement of facts, any co-operation from or attendance at the hearing by the engineer, and consistency with the level of costs in previous decisions. The balance of costs after the orders must be met by the profession itself.[24]

93. In respect of the medical profession, the Court in *Vatsyayann v PCC* said:[25]

> *[P]rofessional groups should not be expected to bear all the costs of a disciplinary regime and that members of the profession who appeared on disciplinary charges should make a proper contribution towards the costs of the inquiry and a hearing; that costs are not punitive; that the practitioner's means, if known, are to be considered; that a practitioner has a right to defend [themselves] and should not be deterred by the risk of a costs order; and that in a general way 50% of reasonable costs is a guide to an appropriate costs order subject to a discretion to adjust upwards or downwards.*

94. Further, in *O'Connor v Preliminary Proceedings Committee* the High Court stated:[26]

[22] Chartered Professional Engineers of New Zealand Act 2002, s 22(4) and IPENZ Disciplinary Regulations, reg 17(3)(g) respectively.
[23] Including *Cooray v Preliminary Proceedings Committee HC* Wellington AP 23/94, 14 September 1995 per Doogue J.
[24] *PCC v Van Der Meer* 1019/Nur18/422P.
[25] [2012] NZHC 1138 at [34].
[26] *O'Connor v Preliminary Proceedings Committee HC Wellington* AP 280/89, 23 August 1990 at [13] per Jeffries J.

> *It is a notorious fact that prosecutions in the hands of professional bodies, usually pursuant to statutory powers, are very costly and time consuming to those bodies and such knowledge is widespread within the professions so controlled. So as to alleviate the burden of the costs on the professional members as a whole the legislature had empowered the different bodies to impose orders for costs. They are nearly always substantial when the charges brought are successful and misconduct admitted, or found.*

95. As noted in submissions, the costs incurred by Engineering New Zealand in carrying out the inquiry into the Masterton Buildings leading to this complaint against Mr O'Connor have been remarkably high when compared with other cases. We acknowledge that the inquiry may have been complex involving the sourcing and review of extensive documentation. We note that much of the work was not undertaken in-house by Engineering New Zealand as would normally be the case. We agree that the input of the expert engineers was valuable and necessary. However, as the breakdown of costs shows, 79% was attributed to the contracted investigator and external legal fees. The bulk of this work would normally be undertaken in-house subject to the availability of resources. Whilst we agree, in principle, that a 40% contribution to costs by the respondent would be appropriate after taking into account the mitigating factors set out by Engineering New Zealand, we consider that it is unreasonable in this case.

96. We have not attempted an audit of the costs, but rather have settled upon a sum of $35,000 as a reasonable and fair contribution to be made by Mr O'Connor. We acknowledge that the balance of the costs will be borne by the profession but consider that on balance the apportionment is fair and reasonable.

97. We have noted Counsel's submission that Mr O'Connor is essentially insolvent and may have difficulty paying. The Disciplinary Committee does not have an obligation to consider a respondent's ability to pay when making orders but does have discretion to do so. However, we do not have sufficient information concerning Mr O'Connor's personal circumstances to take them into account.

Naming

98. In addition to notifying any orders made against an engineer on the register of Chartered Professional Engineers, the Registration Authority must notify the Registrar of Licensed Building Practitioners appointed under the Building Act 2004 of the order and the reasons for it and may publicly notify the order in any other way that it thinks fit.[27]

99. In respect of membership with Engineering New Zealand, the Disciplinary Committee may order that the member be named, the order against the member be stated and the nature of the breach described in the official journal of the Institution or publicised in any other manner as may be prescribed by the Committee.[28]

100. The Act does not prescribe factors the Disciplinary Committee should consider when deciding whether to name an engineer. While we are mindful of the specific legislative test of "desirability" set out in the Health Practitioners Competence Assurance Act 2003, we are guided by the public interest factors

[27] Chartered Professional Engineers of New Zealand Act 2002, s 22(5).
[28] IPENZ Disciplinary Regulations, reg 17(5)(h).

considered by the medical profession when deciding whether to name a practitioner.[29] These include openness and transparency in disciplinary proceedings; accountability of the disciplinary process; public interest in knowing the identity of the practitioner; the importance of freedom of speech; unfairly impugning other practitioners; and that where an adverse disciplinary finding has been made, it is necessary for more weighty private interest factors (matters that may affect a family and their wellbeing, and rehabilitation of the practitioner) to be advanced to overcome the public interest factors for publication.[30]

101. Naming is the starting point and will only be inappropriate in a limited number of circumstances where the engineer's privacy outweighs the public interest. In *Y v Attorney-General*[31] the Court of Appeal explored the principles that should guide the suppression of the names of parties, witnesses, or particulars in the civil context. The starting point is the principle of open justice.[32]

102. The question is then, do the circumstances justify an exception to the principle of open justice. In a professional disciplinary context, a practitioner is "likely to find it difficult to advance anything that displaces the presumption in favour of disclosure".[33] This is because the practitioner's existing and prospective clients have an interest in knowing details of the conduct, as this allows them to make an informed decision about the practitioner's services.[34]

103. Consistent with these precedents, the starting point is that naming of engineers subject to a disciplinary order is the normal expectation. This is because public protection is at the heart of disciplinary processes, and naming supports openness, transparency, and accountability.

104. Having considered the submissions and the public interest in this case, we have decided that Mr O'Connor should be named and our decision should be published in full following determination of the related court claims. We consider there is potential for Mr O'Connor's position to be prejudiced by earlier publication. In ordering interim name suppression until the court processes are concluded, we acknowledge that the time required before the court claims have been determined is unknown and there is public interest in the outcome of the inquiry into the Masterton buildings. However, we consider that the prejudice to Mr O'Connor by naming him before the court matters have been determined outweighs the prejudice to Engineering New Zealand from a delay in publication.

SUMMARY OF ORDERS

105. In exercising our delegated powers, we order that:

a. Mr O'Connor is censured; and
b. Mr O'Connor is to pay a fine of $3500 excluding GST; and
c. Mr O'Connor is to pay $35,000 plus GST towards the costs incurred by Engineering New Zealand in inquiring into Mr O'Connor's conduct.

[29] The presumption in the Health Practitioners Competence Assurance Act 2003 is that hearing shall be in public, but gives the Tribunal discretion to grant name suppression. The test is whether it is "desirable" to prohibit publication of the name or any particulars of the affairs of the person in question and the Tribunal must consider both the interests of any person and the public interest.
[30] *Professional Conduct Committee of the Pharmacy Council of New Zealand v El-Fadil Kardaman* 100/Phar18/424P at [113] – [114].
[31] [2016] NZCA 474.
[32] Ibid at [25].
[33] Ibid at [32].
[34] Ibid.

106. In addition, the Registration Authority will:

a. notify the Registrar of Licensed Building Practitioners appointed under the Building Act 2004 of the order and the reasons for it; and

b. publish the Investigating Committee's and Disciplinary Committee's final decisions on this complaint on its website, in a public press release and in any other communication it considers appropriate following the determination of related court claims.

107. The respondent's name suppression remains in place until the related court claims have been determined.

Jenny Culliford FEngNZ (Ret.)

Chair of Disciplinary Committee

Appendices: Agreed summary of facts, 19 November 2020
Investigating Committee's decisions

APPENDIX A

AGREED SUMMARY OF FACTS

Jenny Culliford
Chair
Disciplinary Committee

Confidential
By email

1 December 2020

Agreed Summary of Facts - Mr O'Connor

1 We write on behalf of counsel for the Registration Authority and Kevin O'Connor in the Own Motion Inquiry in respect of Mr O'Connor (410).

2 Counsel for both parties have agreed on a summary of facts to be filed with the Disciplinary Committee (a copy of that agreed summary of facts is **enclosed**).

3 In the agreed summary of facts:

a Mr O'Connor accepts that he did not act in accordance with standards reasonably expected of a Chartered Professional Engineer or, as applicable, a member of Engineering New Zealand in providing engineering services relating to the design of the Masterton Buildings.

b Mr O'Connor further accepts that the admitted facts establish grounds for discipline.

4 Any decision on whether grounds for discipline exist remains, of course, with the Disciplinary Committee.

5 Counsel for the Registration Authority and Mr O'Connor respectfully request the Disciplinary Committee's indication as to whether any further submissions on that issue are required.

6 In the event that the Disciplinary Committee concludes that grounds for discipline exist, both parties respectfully request an opportunity to make submissions on the question of what penalty, if any, may be appropriate.

Yours faithfully

Hayden Wilson
Chair & Partner
Dentons Kensington Swan

[redacted]

John Morrison
Barrister
Lambton Chambers

[redacted]

Copy: [redacted]

Encl: Agreed Summary of Facts - Mr O'Connor

Before a Disciplinary Committee

Under	the Chartered Professional Engineers of New Zealand Rules (No 2) 2002 and the Chartered Professional Engineers of New Zealand Act 2002
And	the Engineering New Zealand Rules and Disciplinary Regulations
In the matter of	an own-motion inquiry by Engineering New Zealand pursuant to Rule 55(1) of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002, Rule 11 of the Engineering New Zealand Rules and Regulation 13 of the Engineering New Zealand Disciplinary Regulations.
And	**Kevin O'Connor**, of Palmerston North, a Chartered Professional Engineer

Agreed summary of facts

Dated 19 November 2020

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DX SP26517

Solicitor: Hayden Wilson/Madison K. Dobie
[redacted]

May it please the Disciplinary Committee

1 Background

1.1 Mr Kevin O'Connor CPEng CMEngNZ IntPE(NZ) is a Chartered Professional Engineer and a Chartered Member of Engineering New Zealand. He qualified with a Bachelor of Civil Engineering from the University of Canterbury in 1983. He became a Chartered Professional Engineer in 2004. Before that date he had been a Registered Engineer under the previous professional regulation scheme, since 16 December 1988.

1.2 In 2015 concerns were raised with Engineering New Zealand regarding the structure of buildings in Masterton owned by the Masterton Trust Lands (**MTLT**). Those buildings included:

a 57-65 Queen Street

b 57-65 Dixon Street (Lot 9)

c 196-120 Queen Street

d 408 Queen Street and

e Cnr of Dixon and Church Sts

(the Masterton Buildings)

1.3 The Ministry of Business, Innovation and Employment (**MBIE**) commissioned engineering firm GA Hughes & Associates Ltd to undertake a structural assessment of the buildings, who identified concerns and recommended that a Detailed Seismic Assessment be carried out on each building.

1.4 MTLT and MBIE subsequently engaged Holmes Consulting who carried out a "Building Review" that identified concerns about the structural integrity of the buildings.

1.5 Based on these engineering assessments, Engineering New Zealand commenced an own-motion inquiry pursuant to Rule 55(1) of the Chartered Professional Engineers Rules (No 2) 2002 (**Rules**).

1.6 Mr O'Connor was the Chartered Professional Engineer who signed the Producer Statement – Design (**PS1**) for the 57-65 Dixon Street 57-65 Dixon Street (Lot 9), 196-120 Queen Street and

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Cnr of Dixon and Church Sts and, in the case of 408 Queen Street was the Engineering New Zealand member who signed the PS1.

1.7 Although Mr O'Connor did not design the buildings, he accepts that he was responsible for the signing PS1s for each building.

2 Producer Statements

2.1 A Producer Statement is a professional opinion based on the sound judgment and professional expertise of the engineers that issue them. They do not have legal status under the Building Act but are commonly relied on by building consent authorities in determining whether there are reasonable grounds to conclude that engineering work complies with relevant provisions of the Building Code.

2.2 Under the Building Act 2004, section 49(1):

> A building consent authority must grant a building consent if it is satisfied on reasonable grounds that the provisions of the Building Code would be met if the building work were properly completed in accordance with the plans and specifications that accompanies the application.

2.3 Producer Statements therefore provide a building consent authority with confidence that the building work will be constructed in accordance with the Building Code so that it can issue consent for the work. Engineers use PS1s to confirm to building consent authorities that, in their professional opinion, aspects of the building's design comply with the Building Code. There are four different kinds of Producer Statements that can be used to provide verification at different stages in the design and construction process. These are:

a PS1 – design;

b PS2 – design review;

c PS3 – construction; and

d PS4 – construction review.

3 57-65 Queen Street

Design and consenting process

3.1 57-65 Queen Street a single storey building located at 57 – 65 Dixon Street, Masterton. It is constructed with steel portal frames and precast concrete

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panels. The design was completed in 2006. Mr A [redacted] a senior engineer, carried out the detailed design work.

3.2 Mr O'Connor signed a PS1 for the design work dated 6 October 2006, which states:

> I believe on reasonable grounds that subject to:
>
> (i) the site verification of the following design assumptions - safe ground bearing capacity in excess of 100kPa
>
> and (ii) all proprietary products meeting the performance specification requirements,
>
> the drawings, specifications, and other documents according to which the building work is proposed to be constructed comply with the relevant provisions of the building code.

3.3 Prior to signing the PS1, Mr O'Connor was presented with the full design documents and undertook a high level check, but otherwise relied on the structural check carried out by Mr A [redacted] However, Mr O'Connor acknowledged that he did not know the extent of that check, and he confirmed would take the engineer's word that the check had been completed.[1]

3.4 Prior to issuing the building consent, on 24 November 2006 Masterton District Council engaged Spencer Holmes Ltd to "screen the…design for construction and compliance with B1 of the Building Code". On 30 November 2006, Spencer Holmes noted a number of concerns in relation to the engineering design and advised against issuing building consent without a comprehensive peer review. Spencer Holmes also asked for clarifications on a series of technical points.

3.5 KOA Ltd responded to Spencer Holmes' concerns and on 15 December 2006, Masterton District Council issued the building consent.

3.6 Construction of 57-65 Queen Street [redacted] was carried out in 2007.

Design reviews

3.7 In 2016, the structure was reviewed by GA Hughes & Associates Ltd who identified a number of concerns with the structural integrity of the building. GA Hughes & Associates Ltd had concerns about:

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[1] Investigating Committee Decision, paragraph 23

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a the transom supports and method of connection, which may not allow adequate twist restraint required to develop full capacity of the PFC and should be reviewed to ensure there is adequate capacity under the loading standards, NSZ1170.5,

b the restraint of the portal frame rafters given that ductility had been assumed in the design,

c the capacity of the pile assumed to provide fixity at the portal legs, and

d the design calculations do not consider eccentricities as required by the loading standard and there is no consideration of the Cs factor for bracing.

3.8 Holmes Consulting estimated the building strength to be 35-45% NBS. Holmes Consulting also reiterated GA Hughes & Associates Ltd's concerns about the transom beams and the flexural capacity. They noted that: "…[the lack of flexural capacity of the transom beams] means that the embed connections, which have a brittle failure behaviour, are likely to be the limiting component which will have significant impact on the building's ability to sustain seismic demands and would pose a life-safety hazard." Holmes Consulting were also concerned that the maximum frame deflection exceeded the limit set by NZS1170.5.

3.9 Engineering New Zealand engaged Barry Brown CPEng FEngNZ IntPE(NZ) and Stuart George CPEng CMEngNZ IntPE(NZ), two structural engineers, to review the design of the Beaurepaires Building.

3.10 In summary, the experts noted concerns about load paths and fragile foundation/wall connections. Mr Brown considered that the design deficiencies were moderately serious.

4 57-65 Dixon Street (Lot 9)

Design and consenting process

4.1 57-65 Dixon Street (Lot 9)s a single-storey 'L' shaped building with a mezzanine floor, constructed with steel portal frames and concrete precast panels. Its design was completed in 2010.

4.2 Mr O'Connor signed a PS1 in relation to the design of 57-65 Dixon Street (Lot 9) In the PS1, Mr O'Connor certified that:

Subject to:

(i) the site verification of the following design assumptions – safe ground bearing capacity in excess of 100 kPa, and

(ii) all proprietary products meeting the performance specification requirements…

…the drawings, specifications and other documents provided or listed in the attached schedule according to which it is proposed that the building be constructed, comply with the relevant provisions of the Building Code.

4.3 Prior to the signing of the PS1, Mr O'Connor carried out a "quick high level check" of the design documents presented to him by structural engineer, Mr Z [redacted] and relied on the review he had carried out. However, Mr O'Connor acknowledges that he did not know the extent of that check, and he would take Mr Z's [redacted] word that the check had been completed.

4.4 Prior to issuing Building Consent, Masterton District Council engaged Spencer Holmes Ltd who recommended that Masterton District Council accept the PS1 as a basis to issue the Building Consent, and on 30 November 2010, building consent was granted.

4.5 Construction was completed in mid-2011 and an undated PS4 was signed by Mr O'Connor. On 7 November 2011, a Code Compliance Certificate was issued.

Design reviews

4.6 In 2016, the structure was reviewed by GA Hughes & Associates Ltd who identified a number of structural vulnerabilities. GA Hughes & Associates Ltd's concerns included that:

a the roof bracing and load paths were not clear and additional bracing was required between different levels of the roof,

b the transom steel members supporting lateral loads from the concrete wall panels should be reviewed to confirm that they are adequate under the parts provision of the loading standard,

c the base fixings of the concrete wall panels using Reid inserts which will be in the cracked zone for opening actions which would reduce capacity of the inserts, and

d fly bracing has been used but appears to have been omitted in several locations and the restraint of portal legs should be reviewed.

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4.7 In summary, GA Hughes & Associates Ltd was concerned about the bracing and load bearing capacity of the concrete wall panels and roof.

4.8 Holmes Consulting estimated the building strength to be 35-45% NBS and also raised concerns about the bracing as shown on the plans and noted that it was likely that this bracing would yield under moderate earthquake loads rendering it ineffective as a lateral load path. This meant that the primary load system would be the out-of-plane bending of the precast concrete walls. Holmes Consulting noted that the walls subject to biaxial bending would crack which could lead to wall instability.

4.9 Engineering New Zealand engaged Mr Brown and Mr George to review the design of the Carpet Court Building.

4.10 In summary, the experts noted concerns about the bracing and the wall panel/foundation connection, and how the building would carry loads.

5 196-120 Queen Street

Design and consenting process

5.1 196-120 Queen Street is a two-storey commercial/retail rectangular building located at 96-120 Queen Street, and constructed predominantly with steel portal frames and concrete precast panel walls.

5.2 Mr O'Connor signed a PS1, in relation to the design of 196-120 Queen Street In the PS1, Mr O'Connor certified that:

> On behalf of the Design Firm, and subject to:
>
> (i) The site verification of the following design assumptions – safe ground bearing capacity in excess of 100kPa, and
>
> (ii) All proprietary products meeting the performance specification requirements.
>
> the drawings, specifications, and other documents provided or listed in the attached schedule according to which it is proposed that the building be constructed, comply with the relevant provisions of the Building Code.

5.3 Prior to signing the PS1, Mr O'Connor says that he reviewed the design "to the extent warranted by Mr Z's experience". However, Mr O'Connor acknowledges that there was no way of knowing the extent of Mr Z's check and he would have taken Mr Z's word that the check had been completed.

5.4 Prior to issuing Building Consent, Masterton District Council engaged Spencer Holmes Ltd to conduct a "screening review" of the design. Spencer Holmes Ltd recommended that Masterton District Council accept the PS1 as a basis to issue the Building Consent. On 26 July 2011, Building Consent was issued.

5.5 Construction commenced in 2011 and on 4 November 2013, a Certificate of Code Compliance was issued.

Design reviews

5.6 In 2016, the structure was reviewed by GA Hughes & Associates Ltd who identified concerns with the structural integrity of the building, particularly around bracing. GA Hughes & Associates Ltd's concerns included that:

a there was little justification for the selected ductility factor of three,

b bracing details and connections were not compatible with the requirements set out in standards,

c the lateral load resisting system was vulnerable,

d there were no clear connections between bracing elements and precast bracing walls on one side, and

e the capacity of the pile assumed to provide fixity at the portal legs, the transom, fixings and supports may not be sufficient.

5.7 Holmes Consulting estimated the buildings strength to be 35-45% NBS and was concerned about the capacity of the transom beams and their impact on the stability of the precast concrete panel walls.

5.8 Engineering New Zealand engaged Mr Brown and Mr George to review the design of the 196-120 Queen Street

5.9 In summary, the experts were concerned about the lateral resisting system.

6 Cnr of Dixon and Church Sts

Design and consenting process

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6.1 Cnr of Dixon and Church Sts [redacted]s a two-storey commercial/office building located on the corner of Dixon and Church Streets. It has a footprint of two overlapping rectangles, with large pad foundations supporting cantilever precast concrete walls.

6.2 Mr O'Connor signed a PS1 in relation to the design of Cnr of Dixon and Church Sts [redacted] In the PS1, Mr O'Connor certified that:

> On behalf of the Design Firm, and subject to:
>
> (i) Site verification of the following design assumptions safe ground bearing capacity of 300kPa (Gravel)...
>
> (ii) All proprietary products meeting their performance specification requirements.
>
> I believe on reasonable grounds that a) the building, if constructed in accordance with drawings, specifications, and other documents provided or listed in the attached schedule, will comply with the relevant provisions of the Building Code and that b), the persons who have undertaken the design have the necessary competency to do so.

6.3 Prior to signing the PS1, Mr O'Connor conducted a high level review of the design which had been undertaken by Mr Z [redacted] who confirmed he had checked the plans and calculations. However, there was no way of knowing the extent of Mr Z's [redacted] check, and he would take his word that the check had been completed.

6.4 Prior to issuing Building Consent, Masterton District Council engaged Beca Group Ltd to carry out a screening review of the design. Beca Group Ltd noted that two different figures were cited for ground bearing capacity, and they asked for clarification as to which was the correct figure. The PS1 was amended to change the ground bearing pressure.

6.5 The amended PS1 was signed by Mr Y [redacted] CPEng CMEngNZ, as Mr O'Connor was unavailable.

6.6 Beca Group Ltd recommended that Masterton District Council accept the revised PS1 as a basis to issue the Building Consent, On 16 April 2014, the building consent was issued.

6.7 Construction was carried out in 2014. On 11 September 2014, a Certificate of Code Compliance was issued.

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Design reviews

6.8 In 2016, the structure was reviewed by GA Hughes & Associates Ltd who identified structural vulnerabilities. GA Hughes & Associates Ltd noted that

a the use of face loaded walls for bracing together with timber framed walls is unusual and issues of displacement compatibility arise between the relatively flexible face loaded walls lined with Gib,

b the use of Drossbach ducting raised issues of construction quality, and

c there were no fly braces and little restraint for the UB & PFC rafters and beams.

6.9 They also suggested that connections between walls and bracing be subject to a detailed review.

6.10 Holmes Consulting estimated the building's strength to be 55-65% NBS and noted that the roof bracing providing out-of-plane ties to the walls will reach ultimate capacity at 30-35% of current code demand.

6.11 Holmes Consulting also noted that it was likely that the metal strap bracing would yield under moderate earthquake loads, meaning that the primary lateral load system would be the out-of-plane bending of the precast concrete walls. Holmes Consulting noted that concrete walls subjected to biaxial bending would crack on an inclined plane through the wall's thickness, potentially compromising a critical vertical load path for the building.

6.12 Engineering New Zealand engaged Mr Brown and Mr George to review the design of Cnr of Dixon and Church Sts

6.13 In summary, the experts were concerned about the use of precast concrete walls as the primary face load resisting element.

7 408 Queen Street

Design and consenting process

7.1 408 Queen Street is a single-storey commercial/industrial building located at 408 Queen Street Masterton, constructed with a structural steel portal frame and precast concrete panels, with a metal roof.

7.2 Mr O'Connor signed a PS1 on 24 February 2003, in relation to the design of the 408 Queen Street In the PS1, Mr O'Connor certified that:

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I believe on reasonable grounds that subject to:

(i) The site verification of the following design assumptions N/A, and

(ii) All proprietary products meeting the performance specification requirements, the drawings, specifications, and other documents according to which the building proposed to be constructed comply with the relevant provisions of the Building Code.

7.3 Prior to signing the PS1, Mr O'Connor says that the process at KOA Ltd at the time was for a senior engineer to carry out a structural check, which would then be reviewed by Mr O'Connor. Mr O'Connor was not involved in the design development but would have carried out a final high-level review prior to signing the PS1. However, Mr O'Connor accepts that he did not know the extent of the senior engineer's check and would have taken the engineer's word that it had been completed. .

7.4 On 3 and 6 March 2003, changes were made to the dimensions of the building. Mr O'Connor accepts that there is an obvious error relating to the transom in the revision to the plans, but it does not appear that he carried out a review of the final set of plans which incorporated these changes.

7.5 On 12 March 2003, Masterton District Council issued building consent for the 408 Queen St [redacted] Construction was carried out in 2003 and a Certificate of Code Compliance issued on 4 September 2003.

Design reviews

7.6 In 2016, MBIE engaged GA Hughes & Associates Ltd to carry out a high-level structural review. The review identified a number of concerns with the building. In short, GA Hughes & Associates Ltd's concerns included:

a the use of cast in TCM inserts at the cracked zone for opening face loads and doubted that an adequate load path could be achieved between the lower row of inserts and the stabiliser foundation,

b displacement compatibility between the cantilevered wall panels and the steel portal frames, and

c load paths, capacity and connections between the upper gable end wall, the PFC eaves strut/tie and SHS brace on one wall and the concrete panels on the boundary wall.

7.7 Holmes Consulting estimated the building's strength to be 25-33% NBS and also had concerns about the load bearing capacity of the precast panels and noted that the frame would likely suffer buckling as a result of the inelastic displacement demands.

7.8 Engineering New Zealand engaged Mr Brown and Mr George to review the design of 408 Queen Street

7.9 Mr Brown and Mr George provided their advice on the basis of the original drawings, as held on the Council property file, as well as on the basis of the revised design provided by Mr O'Connor.

7.10 Reviewing the original drawings, the experts noted a number of concerns. Mr George identified concerns with the brace details, lack of eaves tie, lack of fly braces, light vertical wall reinforcement, and panel type C fixings. Mr Brown noted concerns with the connections to the precast wall panels and how those connections would manage loads.

7.11 Reviewing the revised design, the experts noted that none of the issues they had identified in respect of the original design had been identified by Mr O'Connor when the drawings were revised. They both considered that Mr O'Connor should have identified the problems with his original design during the revision process and taken steps to modify it.

8 Mr O'Connor's conduct

8.1 Mr O'Connor accepts that the designs of the Masterton Buildings were inadequate and not in accordance with the standards reasonably expected of a Chartered Professional Engineer (or, in the case of 408 Queen Street of a member of Engineering New Zealand).

8.2 Mr O'Connor accepts that he had a responsibility to be satisfied on reasonable grounds that the designs were adequate and in accordance with the requisite standards of the time before signing the PS1.

9 Admission of facts and grounds for discipline

9.1 Mr O'Connor confirms and admits that the facts in this Agreed Summary of Facts are true and accurate and admits that the facts establish the grounds of discipline to the required standard.

9.2 Mr O'Connor accepts that he did not act in accordance with accepted standards reasonably expected of a Chartered Professional Engineer, and, in the case of

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408 Queen Street expected of a member of Engineering New Zealand, in providing engineering services relating to the design of the Masterton Buildings.

9.3 Mr O'Connor further admits that the admitted facts establish grounds for discipline under section 21 of the Act in that Mr O'Connor has performed engineering services in a negligent manner (s 21(1)(c)) and that there are grounds for ordering a disciplinary penalty under section 22 of the Act and, in the case of 408 Queen Street under the rule 11 of the Engineering New Zealand Rules and reg 45 of the Engineering New Zealand Disciplinary Regulations.

Kevin O'Connor

Dated: 27/11/2020

APPENDIX B

INVESTIGATING COMMITTEE DECISIONS

B1: 57-65 Dixon Street

B2: 57-65 Dixon Street (Lot 9)

B3: 96-120 Queen Street

B4: 408 Queen Street

B5: Corner of Dixon and Church Streets

INVESTIGATING COMMITTEE DECISION

OWN MOTION INQUIRY ABOUT KEVIN O'CONNOR — 57-65 DIXON STREET

For release

In accordance with:

Chartered Professional Engineers of New Zealand Act 2002
Chartered Professional Engineers of New Zealand Rules (No 2) 2002

Issued by

Andrew McMenamin CPEng CMEngNZ
Chair of Investigating Committee

Stewart Hobbs CPEng FEngNZ IntPE(NZ)
Dr Sulo Shanmuganathan CPEng FEngNZ
Committee Members

25 March 2019

engineering new zealand
Institute of Engineering Professionals

CONTENTS

BACKGROUND

1. In 2015, Engineering New Zealand[1] was made aware of concerns surrounding six buildings in Masterton owned by Masterton Trust Lands Trust (MTLT). The concerns were raised by a Chartered Professional Engineer (CPEng).

2. A building located at 57-65 Dixon Street, was one of the buildings identified.

3. The concerns were about the structural integrity of the buildings. Because Engineering New Zealand has no jurisdiction over physical assets, Engineering New Zealand brought the concerns to the attention of the Ministry of Business, Innovation and Employment (MBIE), as the relevant regulatory authority.

4. MBIE subsequently commissioned GA Hughes & Associates Ltd to carry out a structural review of the buildings,[2] which identified concerns with the buildings and recommended a Detailed Seismic Assessment be carried out. Following the receipt of this report, MTLT and MBIE commissioned a Detailed Seismic Assessment to be carried out by Holmes Consulting, which identified concerns about the structural integrity of the six buildings. MTLT subsequently commenced remedial works on some of the buildings.

5. In light of the findings of these two reports, Engineering New Zealand decided that it needed to act on this information to determine if there is an issue with the engineering design of these buildings or not and, if so, what that means. It was decided that the best way to do this was by way of an own-motion inquiry pursuant to Rule 55 (1) of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002 (the Rules).

[1] On 1 October 2017 IPENZ changed its name to Engineering New Zealand. Accordingly, I will refer to Engineering New Zealand throughout this report.

[2] One of the buildings that Engineering New Zealand was initially notified about was different from the six ultimately chosen by MBIE to assess further.

SCOPE OF INQUIRY

6. The scope of the Inquiry was to conduct an initial investigation into:

   "The circumstances relating to the design, design review and construction monitoring of six buildings in Masterton:

   - to assess whether the Chartered Professional Engineers involved have provided engineering services in accordance with accepted standards; and
   - to learn and advise on any engineering performance and practice improvements, if necessary."

7. Kevin O'Connor & Associates (KOA Ltd) was the engineering firm involved in the engineering design of all six of the buildings. Kevin O'Connor CPEng, CMEngNZ[3], IntPE(NZ)[4] was identified as the Chartered Professional Engineer who signed the PS1 for the building at 57-65 Dixon Street.

8. The building was designed and built in 2006.

9. The issue being considered with respect to Mr O'Connor is:

   - Whether Chartered Professional Engineer Kevin O'Connor provided engineering services relating to the engineering design work on 57-65 Dixon Street in accordance with accepted standards.

INVESTIGATION

10. Following an initial investigation this matter was referred to an Investigating Committee for formal investigation. The Investigating Committee is:

    Andrew McMenamin CPEng CMEngNZ (Chair)

    Dr Sulo Shanmuganathan CPEng FEngNZ

    Stewart Hobbs CPEng FEngNZ IntPE(NZ)

11. Information considered by Engineering New Zealand for the purposes of the investigation included information from:

Kevin O'Connor	Respondent/engineer
Kevin O'Connor & Associates Ltd	Engineering company
Masterton District Council	Local authority
Masterton Trust Lands Trust	Building owner
GA Hughes & Associates Ltd	Consulting structural engineering company
Holmes Consulting	Consulting structural engineering company

12. Independent expert advice was obtained from Barry Brown CPEng, FEngNZ[5], IntPE(NZ) (**Appendix A**) and Stuart George CPEng, CMEngNZ, IntPE(NZ) (**Appendix B**).

[3] Chartered Member of Engineering New Zealand.

[4] A member of the New Zealand section of the International Professional Engineers.

[5] Engineering New Zealand Fellow.

INFORMATION GATHERED

13. Masterton Trust Lands Trust (MTLT) engaged Proarch Architects Ltd to design a new building at 57-65 Dixon Street.

14. In August 2006, Proarch Architects Ltd engaged Kevin O'Connor & Associates Ltd (KOA Ltd), on behalf of MTLT to provide the engineering design for the development. While the contract for services was between MTLT and KOA Ltd, MTLT told Engineering New Zealand that Proarch Architects Ltd acted as the project manager and all communications with KOA Ltd and the Masterton District Council went through Proarch Architects Ltd. The short form contract was signed by Kevin O'Connor on 7 August 2006.

BUILDING DESIGN

15. The building is a single storey building, constructed with steel portal frames and precast concrete panels. The foundations are concrete slab on grade, with slab thickenings running along all four walls.

ENGINEERING DESIGN

16. Mr O'Connor told Engineering New Zealand that his role in the development of this building was to "set up the job which included pricing, as well as doing some basic analysis of the job to determine who would do the design work".

17. Mr O'Connor said that the job was allocated to a senior engineer who "conducted the detailed design work and checking of the plans". Mr O'Connor told the Investigating Committee that his employee was a very experienced engineer and he relied on him to carry out all the detailed checks on the designs and calculations.

18. Engineering designs were completed in September 2006 (design sheets S1-S10). According to the issued set of plans they were designed by Kevin O'Connor (and his senior engineer) but drawn by a draughtsperson.

19. The engineering calculations for this design are not initialled or dated.

20. Mr O'Connor told Engineering New Zealand that the structural analysis software, Turboframe, was used for the designs.

21. Mr O'Connor signed a Producer Statement — PS1 — Design (PS1) dated 6 October 2006 which states:

> "I BELIEVE ON REASONABLE GROUNDS that subject to:
>
> (i) the site verification of the following design assumptions – safe ground bearing capacity in excess of 100 kPa.
>
> and (ii) all proprietary products meeting the performance specification requirements,
>
> the drawings, specifications, and other documents according to which the building work is proposed to be constructed comply with the relevant provisions of the building code." *(emphasis in original)*

22. Mr O’Connor told Engineering New Zealand that, in relation to the extent of the quality assurance process in place, “the plans would have been subject to a normal dimensional check and a structural check by the designer before being issued”. Prior to signing the PS1 Mr O’Connor said that he “reviewed the design done by [the senior engineer] to the extent warranted given his experience and expertise”. Mr O’Connor stated:

> “A senior engineer, was allocated to do the design work. This engineer had 30 years of experience, most of which was as the Structural engineer for the Palmerston North City Council. During this project, I managed resourcing of appropriate personnel to ensure appropriate staff responded to matters raised in the normal review process.”

23. Mr O’Connor said his checks would have included calculations and design, but he did not provide any additional or specific details about what his review included in this case. Furthermore, Mr O’Connor told the Investigating Committee that he relied on the detailed structural check that would have been carried out by the senior engineer. Mr O’Connor said that there is no way of knowing that the senior engineer had completed their check or the extent of that check and would take the engineer’s word that the check had been completed.

24. Mr O’Connor told the Investigating Committee that all the documentation would be provided to Proarch Architects Ltd as they managed all the correspondence with the Council.

25. On 25 October 2006, an application for Building Consent was submitted by Proarch Architects Ltd. This was formally recorded as being received on 27 October 2006.

STRUCTURAL CONCEPTS

26. Steel UB portal frames stabilise precast concrete wall panels.

27. According to the GA Hughes & Associates Ltd report, gravity loads are taken by the steel purlins to steel UB rafters which are supported by UB portal legs at the front and back of the building.

28. Lateral wind and earthquake loads across the building are resisted by the portal frames which have fixed bases.

29. The PFC transom member are connected to the portals with a web plate and bolts at each end.

30. Fire face loads are provided for by cantilevered concrete panels above the foundation.

SCREENING REVIEW

31. On 24 November 2006, Masterton District Council wrote to engineering firm Spencer Holmes Ltd requesting a “screening review”. The request stated:

> “Please screen the engineers design for construction and compliance with B1 of the building code.
>
> Please advise if the portal columns are integral to the support of the pre-cast concrete panels and should be fire rated?”

32. On 30 November 2006, Spencer Holmes Ltd wrote to Masterton District Council noting a number of concerns in relation to the engineering designs. Spencer Holmes requested the following information:

“**1. Purlins**

Details of fixings for purlins require consideration of

a) Load on the eaves PFC on minor axis, and

b) Eccentricity of load from the DHS through to the PC panel under both vertical and horizontal loadings

2. Slab Control Joints

These should be shown.

3. PC Panel Reinforcing

Anchorage of the vertical bars in the PC panels requires to be below the Reid Bar Footplate in accordance with the Reid manual details.

4. Piles

Design for the pile foundations is not included, nor the base plate for the portal frame.

5. Portal Base Fixity

Base moment fixity of the portals requires confirmation of adequate restraint by the slab and PC panels against outward movement.

6. PC Panel Fixings

Clarification is required of PC panel/panel and panel/UB connections at Grids D, D and E. All connections shown appear to be panel/panel – not to the UB. How is lateral support then to be provided?

7. Specifications

There is no specification.”

33. The letter goes on to state: “We are of the opinion that there are significant items arising from the initial screening that we would caution the Masterton District Council against issuing a Building Consent in respect of the application without comprehensive peer review.”

34. Mr O’Connor provided Engineering New Zealand with the KOA Ltd file copy of this letter which is annotated with names next to each of the questions asked by Spencer Holmes Ltd. Mr O’Connor stated that these annotations indicate “management of appropriate staff to respond to review questions.” Mr O’Connor also provided a copy of an internal email dated 4 December 2006 from the senior engineer to himself in which he provides responses to the questions in the Spencer Holmes letter.

35. In the email the senior engineer stated:

“1. Purlins a) The PFCs at the eaves are supported every metre on the minor axis. Where there is no panel the PFC has been strengthened to allow for this.

b) Any induced eccentricities is minor and the detail is a perfectly standard one and has never caused problems in the past. …"

36. The senior engineer also responded to the queries about the slab control joints, panel reinforcing, piles, portal base fixity, and PC panel fixity referring to details on the plans.

37. The Masterton District Council records indicate information about the Spencer Holmes Ltd review was received on 13 December 2006, although it does not list who information was requested or received from, nor what information was requested.

38. We have not found any documentation on the Council file provided to Engineering New Zealand to indicate how the Council responded to the concerns raised by Spencer Holmes during the screening review or the process followed to consider KOA Ltd's responses to those concerns.

39. On 14 December 2006, Masterton District Council wrote to KOA Ltd advising that it was about to issue building consent for the development and noted that the consent would be issued with the condition that a PS4 construction is supplied. The letter requested confirmation that KOA Ltd had been engaged "to carry out construction monitoring of the structural aspects to ACENZ/IPENZ level CM3[6] to enable the issue of a producer statement PS4 — construction review on completion of the work."

40. We have not found any correspondence from KOA Ltd on file in response to the Masterton District Council's letter. However, on 15 December 2006 Building Consent was issued with the following condition listed:

> "1. The issue of Code Compliance Certificate for this consent will be reliant on the issue of a PS4 from the design engineer covering inspection of all aspects of their design."

CONSTRUCTION

41. Construction was carried out in 2007.

42. According to the Masterton District Council records, a foundation inspection was completed on 16 January 2007, and on 21 February 2007 a pre-lining inspection was completed which included a bracing check. No issues relating to the construction were identified.

43. The short form contract signed by Mr O'Connor on 7 August 2006 lists under the scope and nature of the services that construction monitoring would be carried out to a CM3, although the cost proposal for this has been crossed out on the contract. In his statement to Engineering New Zealand Mr O'Connor stated: "Please note construction observation was not required. Some observation was done post the initial engagement." Further to this Mr O'Connor stated:

> "KOA [Ltd] offered to conduct construction observation to level CM3. Initially construction observation was declined by MTLT. Refer to strike through item on Short Form [contract] …. The architect subsequently asked KOA to do limited construction observation consisting of three inspections. Our records show KOA [Ltd] did limited inspections on the foundations, on the initial panel and the structural steel erection, as well as a pre-pour inspection of the concrete floor."

44. No PS4 was provided for this building. In his statement to Engineering New Zealand Mr O'Connor said that "No PS4 was requested. Our records show none was issued …"

45. On 3 April 2007, a Code Compliance Certificate was issued.

[6] CM3 monitoring is defined as: "Review, to an extent agreed with the client, random samples of important work procedures, for compliance with the requirements of the plans and specifications and review important completed work prior to enclosure or on completion as appropriate."

SUBSEQUENT REVIEWS OF DESIGNS

46. On 4 August 2014, MTLT wrote to Proarch Architects Ltd, noting that it had become aware of engineering design issues relating to another project that KOA Ltd had provided engineering design for. MTLT requested advice as to whether similar issues may also apply to any of its buildings that KOA Ltd had been involved in. This letter was forwarded to KOA Ltd. On 23 September 2014, Mr O'Connor responded directly to MTLT stating that he had carried out a desktop review of all the relevant buildings.[7] He stated that following his review, in his opinion, "the buildings … [have] been properly designed and detailed".

47. On 3 November 2014, MTLT responded that a review by Mr O'Connor or any current KOA Ltd employee did not provide the level of necessary comfort from an independent perspective. MTLT requested advice on how this could be achieved and whether it should consider requesting a peer review.

48. On 21 November 2014, Mr O'Connor responded stating that "I fully understand that KOA could be seen to have a financial interest which could be seen as impairing the personal judgement of the reviewers" and recommended that MTLT seek a detailed seismic assessment.

49. It does not appear that any further action was taken by MTLT at that time.

GA HUGHES & ASSOCIATES LTD

50. As noted above, in June 2016, GA Hughes & Associates Ltd undertook a high level structural review of the building. The information considered as part of the Hughes review included the Masterton District Council property file, including the engineering calculations and drawings. A site visit and visual inspection of the building was also carried out. GA Hughes & Associates Ltd also undertook an Initial Investigation Procedure (IEP) for the building, which is a quick assessment method based on factors such as the buildings age, the type of materials used, and the construction type, used to identify if a building is potentially earthquake prone.

51. In summary, the review identified concerns with the structural integrity of the building. The report summary stated:

> "The transom supports at the portal frame legs and the method of fixing to the concrete wall panels may not allow adequate twist restraint required to develop full capacity of the PFC. This member should be designed under the parts provision of NZS1170.5.
>
> "While there are two fly braces to the portal frame rafters we have concerns about the restraint of the portal given that ductility has been assumed in the design.
>
> "There are concerns about the capacity of the pile assumed to provide fixity at the portal legs.
>
> "The design calculations do not consider eccentricities as required by the loading standard and there is no consideration of the Cs factor for bracing.
>
> "The adequacy of the transoms, their fixings and connections should be reviewed to determine if there is adequate capacity under the parts provisions of the loading standards."

[7] This included 57-65 Dixon Street.

HOLMES CONSULTING

52. In September 2016, Holmes Consulting undertook a Detailed Seismic Assessment of the building, “with review of the structural capacity of vulnerable details in the […] buildings in question as identified by previous reporting by G.A. Hughes.”

53. The report noted that the “principal elements of our scope are to provide an independent review of load paths and configuration, as well as key detailing that may affect the seismic capacity of the buildings.”

54. The review estimated the building’s strength to “35-45% of the strength of an equivalent new building designed to AS/NZS 1170.5:2004.” It stated:

    > “The governing element is the embed connections between the precast panels and the upper transom beam, with the embedded anchors having insufficient pull-out/concrete cone failure capacity to sustain the out-of-plane wall loading. The transom beams, a PFC steel section, yield at a demand lower than 35% however we have allowed that they will have some ability to sustain inelastic deformations beyond yield even if this results in some lateral flexural buckling and torsion. This means that the embed connections, which have a brittle failure behaviour, are likely to be the limiting component which will have significant impact on the buildings ability to sustain seismic demands, and would pose a life-safety hazard.”

55. It is also noted in the report that the precast concrete panels along the North elevation “are likely to be unstable in a moderate to large earthquake” due to the 200 PFC transom beams having “insufficient flexural capacity to carry large out-of-plane loads” and “connections between the transom beams and precast panels occur on only one side of the beams”. It stated: “In general, PFC sections are not ideal to resist torsional moments, and particularly not when considering the spans and loads present in the building.”

56. The report also found that the “maximum frame deflection was found to be in excess of 6%”, noting that this exceeds the 2.5% limit set by NZS1170.5. It noted, however, that “before these large displacements can develop it is expected that secondary load-paths via the purlins and rafters (out-of-plane bending) and the roof cladding itself will limit the displacements to less than that calculated for a bare portal frame”.

57. It also noted that the calculated drift value included allowance for partial fixity at the base of the portal frame columns and “[a]s a result, a relatively large flexural moment is transferred from the portal frame columns into underlying concrete piles”. It noted that because the piles are only 2 m in length “it is likely that the soil surrounding the piles would fail in bearing under anticipated seismic loads, resulting in the piles ‘rocking’ back-and-forth within their pockets”.

58. Lastly, it is noted that the precast concrete walls along the east and west elevations are supported through an eccentric connections to roof purlins “that would likely result in bolts connecting the purlins to the wall tearing through the purlins”, although it is noted that this would only occur at large displacements and unlikely to occur before pull-out of the transom connections.

EXPERT ADVICE

59. Engineering New Zealand engaged two independent expert structural engineers — Stuart George and Barry Brown — to provide advice on this matter. They were asked to advise whether, in their opinion, the engineering designs were produced in accordance with accepted standards at the time and consistent with what a reasonable engineer, in the same circumstances, would likely have done. They were not asked to undertake a seismic assessment of the building and neither have undertaken an evaluation of the seismic performance rating. This is a key difference in the focus of the advice provided by Mr George and Mr Brown compared to the assessments undertaken by GA Hughes & Associates Ltd and Holmes Consulting – the Hughes and Holmes reports are instead concerned with the building's current performance.

60. The information Mr Brown and Mr George considered in forming their opinions included the Masterton District Council property file, Mr O'Connor's submissions, as well as the GA Hughes & Associates Ltd and Holmes Consulting reports.

61. In assessing whether the overall design met acceptable standards Mr George stated:

> "The definition of meeting acceptable standards is difficult to define. My best analogy would be marking a project at Engineering School, it is inevitable that [there] will be a wide variation in marks, and unlikely that anyone would score 100%. Even with the benefit of years of experience and an engineering registration process engineering knowledge and judgement varies."

62. Mr George gave the design for this building an overall grade "B", noting in particular concerns relating to the load paths, as well as the foundation/wall connections, which he considered appeared fragile.

63. Mr Brown identified concerns relating to the precast lintel tie system over the workshop doorways. He also noted concerns relating to the foundation/wall anchored joint relating to the same doorways. Mr Brown considered that overall, the design deficiencies were moderately serious.

64. See **Appendix A** and **B** for a full copy of their reports.

RESPONSE FROM MR O'CONNOR

65. In a statement to Engineering New Zealand dated 29 May 2017 Mr O'Connor stated:

> "In managing the engineering service (design & limited construction observation), I feel the project resourcing, including a highly-experienced designer in combination with the council review process culminated in a compliant building."

66. Mr O'Connor stated that this has been confirmed through a review "by a senior KOA [Ltd] engineer using Equivalent Static Analysis (ESA)[8]", which confirmed "that the building whilst being a code minimum building was nonetheless compliant [with the relevant design standards at the time]". He also advised that a further review, carried out by KOA Ltd, using a "3-Dimensional Non-Linear Push-over Analysis (NLPA)", "conducted using the loads requirements of NZS 4203 and NZS 3404 applicable

[8] ESA is a method used to analyse the seismic performance of a building.

at the time the building was designed" confirmed "the design of the building complies to 100% NBS using ductility factor of 1.25."

MR O'CONNOR'S COMMENTS ON THE GA HUGHES & ASSOCIATES LTD REPORT

67. In relation to the comments of the G.A. Hughes & Associates Ltd report, Mr O'Connor noted that Mr Hughes assessed the building as being irregular in shape when in fact it was rectangular. Mr O'Connor stated that "[t]his is significant because it allows a 30% reduction in capacity" resulting in a %NBS of 55% and 75% rather than 79% and 100% respectively for each direction.

68. In relation to the comment that the portals, transoms and panel fixings need to be checked in respect of the parts provisions of NZS 1170.5, Mr O'Connor submitted that "the building was designed to NZS 4203 and codes at the time did not require this" and that it was common practice for precast panels not to be considered a "part" and therefore would not have been designed as such.

69. In relation to the concern regarding the adequacy of the transoms and fixings Mr O'Connor stated:

    "The transom connections to the panels are adequate and prevent collapse.

    "The transoms are adequate for the load applicable at the time of design.

    "The transoms are fully restrained on grids A and F.

    "The transoms on grid 3 could easily be further restrained to comply with NZS 1170 by adding a simple angle fixed to the unrestrained flange of the transom and the purlin immediately adjacent to it."

MR O'CONNOR'S COMMENTS ON THE HOLMES CONSULTING REPORT

70. In relation to the comments in the Holmes report, Mr O'Connor noted that it had assessed the building in terms of NZS 1170 but states that "the building was not designed to this code". Mr O'Connor submitted that the "transoms are suitable to support the loads applied" and that these elements "comply with the code requirements applicable at the time of design".

71. In relation to the comment that the embed connections between the transom beams and precast panels pose a life-safety risk, Mr O'Connor stated: "it is unreasonable to hold KOA to account for recent knowledge that has come to light post design and construction and after the poor performance of the cast in fixings post [Canterbury] earthquakes".

72. In relation to the comment that the drift exceeded 2.5%, Mr O'Connor noted that the building was not designed to NZS 1170. He also commented that, in his opinion, the 2.5% limit is an "inter-storey deflection limit" and does not apply to single storey buildings.

73. In relation to the adequacy of the roof bracing, Mr O'Connor stated that "[t]his has never been shown to be a problem in causing building failures". In relation to the adequacy of the pile foundations, Mr O'Connor submitted that "the piles are founded in stiff gravels and when boring such piles in gravel the holes widen considerably" and, as a result, "their capacity will greatly exceed the demand". Mr O'Connor submitted that the "limiting load capacity in the ground will be at least 200 Kn.m" and the "hinging at the portal base would occur first before [any] rotation of the pile in the ground". Mr O'Connor submitted that these comments are based on the assumption that the ground is soft and that the ground is in fact founded in "stiff river gravels" and that they have been designed "assuming a

lateral soil capacity of 300 KPa" and therefore it is "extremely unlikely" that they will rock in the ground as suggested in the report.

RESPONSE TO PROVISIONAL DECISION

74. In response to the Investigation Committee's provisional decision Mr O'Connor noted that in its decision the investigation Committee is considered his actions in relation to the signing of the PS1, but has not identified what accepted standards in relation to the use of PS1s are. Mr O'Connor referred to Engineering New Zealand's Practice Note 1 Guidelines on Producer Statements,[9] noting it states that "limited standardised practice has developed" in relation to the use of PS1s and that a producer statement is "not a product warranty or guarantee of compliance".

75. Further to this, Mr O'Connor reiterated his view that, based on the information he had available when signing the PS1, he had reasonable grounds to believe that the designs complied with the Building Code. Mr O'Connor stated:

> "In signing the PS1 I confirmed that I had a reasonable basis for thinking that, in my opinion, the designs complied with the Building Code. I did have a reasonable basis for thinking the designs complied with the Building Code. … [i]t was my practice to check aspects of the calculations in the designs, and to place weight on the fact the designs had been prepared and/or reviewed by an engineer with suitable experience and the person doing this work had 20 years experience as an engineer and the building is a relatively low risk structure smaller than most houses."

76. Mr O'Connor noted that there was no requirement for him "to exhaustively review the designs and calculations for the purpose of preparing the PS1".

77. In addition, Mr O'Connor expressed concern regarding the conclusion in Mr Brown's advice report of 30 April 2018, in which Mr Brown stated: "if the output PS1 for design … signed off by the CPEng/Principal is defective in terms of its statement … the author must accept responsibility arising from that." In relation to this statement Mr O'Connor stated:

> "This is not the inevitable consequence of a defective design, if the author of the producer statement-took reasonable steps before signing it. A producer statement states that the author believes on reasonable grounds etc… Thus the issue is whether reasonable steps were taken, but in hindsight if those steps are found to reveal an error may have occurred that does not make the process unreasonable."

78. Mr O'Connor stated that the environment and standards have changed significantly since the Canterbury earthquakes. Mr O'Connor stated:

> "The environment we work in now as engineers and the standards expected have significantly changed because of the scrutiny applied to the profession since the Christchurch earthquakes. I believe if such scrutiny was applied to engineering work prior to this time, many practices and design details when judged by today's standards, would be discovered and deemed deficient."

79. Mr O'Connor reiterated his submission that many of the details that have been identified as being deficient in these cases were commonplace at the time the buildings were designed.

[9] See: www.engineeringnz.org/resources/practice-notes-and-guidelines/

DISCUSSION

INVESTIGATING COMMITTEE'S ROLE AND THE LEGAL CONTEXT

80. Professional disciplinary processes primarily exist to protect the public, uphold professional standards, and maintain public confidence in the profession and its regulation. They do this by ensuring that members of the profession adhere to certain universal (or accepted) professional standards.[10]

81. Our role in investigating this professional disciplinary process is to determine whether there are grounds to dismiss the matter as set out in rule 57 of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002 (the Rules). If none of these grounds to dismiss the matter apply, then the matter must be referred to a Disciplinary Committee in accordance with rule 60(a) of the Rules.

82. In order to determine whether there are grounds to dismiss the matter, we have investigated whether Mr O'Connor provided engineering services relating to the engineering design work on the building at 57-65 Dixon Street in accordance with accepted professional standards at the time the building was designed. We are not concerned with whether the building strictly complied with the building code or the building's current seismic rating – our assessment is whether Mr O'Connor acted reasonably at all stages of his involvement in the design of this building, including when he reviewed the design and signed the PS1.

83. The legal test we need to assess Mr O'Connor's actions against is whether he acted reasonably and in accordance with what a reasonable body of his peers would have done in the same situation (i.e. did he act in accordance with accepted standards).

84. If the evidence is that Mr O'Connor acted in accordance with accepted standards then we can dismiss the matter.[11] If the evidence is that Mr O'Connor did not act in accordance with accepted standards then we need to assess how significant his departure from accepted standards was – if it was minor, we may dismiss the matter as insufficiently grave to warrant further investigation;[12] if it is more than minor, and no other ground for dismissal applies,[13] we are required to refer the matter to a disciplinary committee.

85. The issue we have specifically considered in this case is whether in signing the PS1 Mr O'Connor was acting in accordance with accepted professional standards at the time the building was designed.

REVIEW AND SIGN OFF OF THE DESIGNS BY WAY OF PS1

Adequacy of the design

86. In his statement dated 29 May 2017, Mr O'Connor submitted that the building at 57-65 Dixon Street was a "compliant building" and that the building is safe, which he says has been confirmed through a number of reviews and checks conducted by senior KOA Ltd engineers using different methods

[10] *Dentice v Valuers Registration Board* [1992] 1 NZLR 720 (HC).

[11] Rule 60(a) of the Rules.

[12] Rule 57(ba) of the Rules.

[13] There are other grounds in the legislation for dismissing a complaint, including: where the subject matter of the complaint is trivial; the complaint is frivolous or vexatious or is not made in good faith; where the person alleged to be aggrieved does not wish action to be taken or continued; where the complainant doesn't have a sufficient personal interest in the subject matter of the complaint; or an investigation is no longer practicable or desirable given the time elapsed since the matter giving rise to the complaint (Rule 57 of the Rules). In our view, given the nature of the Inquiry, we cannot apply any of these grounds to reasonably dismiss this matter.

(although he has not provided us with full copies of these). We also note Mr O'Connor's submission regarding the assessments carried out by G.A. Hughes & Associates Ltd and Holmes Consulting, and in particular that these assessments were based on standards that the building was not designed to.

87. While it was appropriate that G.A. Hughes & Associates Ltd and Holmes Consulting considered current standards in their reviews, our focus is on whether Mr O'Connor complied with accepted practice at the time. Although NZS 1170.5 was published in 2004, it did not come into effect until 1 December 2008. Therefore NZS 4203 was the relevant standard in relation to the building. Both Mr George and Mr Brown considered the design against NZS 4203.

88. While both Mr George and Mr Brown considered that the design was generally well developed with clear load paths, they considered that there were concerns relating to the precast lintel tie system and foundation/wall connections. Mr Brown advised:

> "Whilst the base connection on the long walls might have adequate capacity, we believe the short return wall (grid 2/E-F) which provides longitudinal stability to the S-W (door opening) side of the workshop may have insufficient combined tensile/shear capacity in its RB16 connections to withstand severe seismic design action."
>
> "The sufficiency of the door lintel tie fixing may require further investigation to ensure that there is sufficient capacity in this system to withstand a severe seismic event without collapse."

89. Mr Brown considered these issues were moderately serious. Mr George also identified concerns with base connections.

90. In response to the provisional decision, Mr O'Connor submitted that many of the details that have been identified as being deficient were commonplace at the time of the building's design and are the same as those used by Holmes Consulting in recent examples. Mr O'Connor stated:

> "The environment we work in now as engineers and the standards expected have significantly changed because of the scrutiny applied to the profession since the Christchurch earthquakes. I believe if such scrutiny was applied to engineering work prior to this time, many practices and design details when judged by today's standards, would be discovered and deemed deficient."

91. As noted above, our experts were asked to advise whether, in their opinion, the engineering designs were produced in accordance with accepted standards at the time the building was designed – 2006 in the case of 57-65 Dixon Street– and consistent with what a reasonable engineer, in the same circumstances, would likely have done.

92. We note that both the experts considered all the information provided to Engineering New Zealand by Mr O'Connor, including his submission of 29 May 2017, as well as a summary of the Investigating Committee's interview with Mr O'Connor on 26 March 2018.

93. Mr O'Connor has provided no new information. Accordingly, guided by our experts, we confirm our concern that there are a number of deficiencies with the original engineering design of this building that indicate it was not of an adequate standard, taking into account the year that it was designed.

Should Mr O'Connor have signed the PS1?

94. Chartered professional engineers use PS1s to confirm their professional opinion that aspects of a building's design comply with the Building Code. Their intent is to signal to a building consent authority (BCA) that certain design work has been done (or overseen/supervised) by a practitioner who is competent to perform the defined work.[14]

95. Although they have no legal status under the Building Act 2004, PS1s are intended to provide BCAs with information to establish that there are reasonable grounds for the issue of a building consent. When used properly, they give BCAs confidence that certain building work will be constructed to meet the Building Code.

96. Chartered professional engineers should be aware that BCAs are likely to rely on producer statements to some extent, and therefore should be mindful that responsibilities and potential liabilities may arise from signing them.[15]

97. In signing the PS1 for the building Mr O'Connor was confirming his professional opinion that aspects of the building's design complied with the Building Code. Accordingly, by signing the PS1 Mr O'Connor assumed responsibility for the building's design.

98. Therefore, the next question for us to consider is whether it was reasonable for Mr O'Connor to sign off the building's design by way of PS1, or whether he should have identified and responded to the issues identified by our experts, as noted above, before providing his sign off. In other words, the question for us is whether Mr O'Connor carried out his part in the review and sign off process in a careful and competent manner and in accordance with accepted professional standards.

99. The starting point for this assessment is to consider Mr O'Connor's role in the review and sign-out process.

100. In his statement dated 29 May 2017, Mr O'Connor told Engineering New Zealand that his role in the design and development of the building was limited to acting as the client interface and providing the final review and sign off of the design by way of PS1. Mr O'Connor said that he "reviewed the design done by [Terry French] to the extent warranted given his experience and expertise". Mr O'Connor said his checks would have included a review of the calculations and designs, but said that he was guided by the senior engineer carrying out their detailed structural check of the designs. Mr O'Connor said that the senior engineer had over 30 years' experience and much of it working for a council. However, during an interview with the Investigating Committee he said that he had no way of confirming the check had been completed or to what extent - he was reliant on the senior engineer's word.

101. Mr O'Connor noted in his response to the provisional decision that there is no standard practice in relation to the use of PS1s and he refers to Engineering New Zealand's Practice Note 1 *Guidelines on Producer Statements* (the Practice Note) to support this. He also said that there was no requirement for him to "exhaustively review the designs and calculations for the purposes of preparing the PS1". Mr O'Connor submitted that based on his review and assurance provided by the senior engineer involved he had "a reasonable basis for thinking the designs complied with the Building Code".

[14] Engineering New Zealand Practice Note 1 *Guidelines on Producer Statements* (January 2014), section 3.1.

[15] Ibid.

102. The part of the Practice Note that Mr O'Connor refers to in support of his argument about standard practice states: "By the same token there is no provision for BCAs to require a producer statement, as of right. As a result of their non-mandatory status, limited standardised practice has developed. There is currently such a wide variation in the way BCAs accept and/or rely on producer statements that there is a resultant degree of confusion, frustration and inefficiency amongst practitioners and BCAs." We interpret the Practice Note to be saying that the lack of standardised practice related to the extent by which producer statements are used by BCAs; not the extent of engineering practice with regard to the level of review a chartered professional engineer should carry out before signing off a PS1 – which is the issue in this case. We consider that, in 2006, there would be a clear expectation from Mr O'Connor's peers that a chartered professional engineer signing off a design by way of PS1 would take sufficient steps in their review of a design to satisfy themselves that they had reasonable grounds for signing the PS1.

103. We accept there was no requirement to undertake an "exhaustive" review himself. However, as noted above, in signing the PS1 Mr O'Connor was confirming his professional opinion that aspects of the building's design complied with the Building Code. Regardless of Mr O'Connor's trust in the experience and competence of the designer or the senior engineer who carried out the checks, as a CPEng engineer and the person signing out the designs, Mr O'Connor had a responsibility to carry out an adequate review himself, and not just rely on the reviews that may or may not have been carried out at other steps of the system. As noted by Mr Brown:[16]

> "if the output PS1 for design … signed off by the CPEng/Principal is defective in terms of its statements (and irrespective of the reliance placed on verbal advice given by a Senior Engineer), the author must accept the responsibility arising from that. Put another way, the process that the CPEng/Principal used in reaching his/her judgement regarding the sufficiency of building design documentation is irrelevant if the statement is defective."

104. We note that in response to the provisional decision Mr O'Connor argued that if the author of a PS1 took reasonable steps to check the design met the relevant standards, if an error is later identified, "that does not make the process unreasonable".

105. We agree with Mr O'Connor that the issue is whether he took reasonable steps to check the designs.

106. Mr George advised that the issues in the design should have been identified during the checking stages of the design development. There is no evidence that they were.

107. Overall, when weighing up the advice from the experts, it is our opinion that Mr O'Connor did not act in accordance with accepted standards in signing off this design by way of a PS1. As noted by Mr Brown, the issues identified may affect the building's capacity to withstand a severe seismic event. In our view, this is a significant design issue that needed to be carefully considered. We consider that, as the CPEng signing off the design, Mr O'Connor should have identified the issues noted by our experts and, in the very least, taken additional steps in light of those issues to satisfy himself that the building met relevant standards before signing the PS1. That may have included, for example querying this with the design engineer or senior reviewing engineer. There is no evidence that he did identify and respond to these issues.

[16] See report dated 30 April 2018.

108. Further, in our view, a reasonable engineer would document any process of checking and clarifying aspects of a design at the sign off stage and included documentation to support their assertions.

109. In our opinion, a reasonable body of Mr O'Connor's peers would likely consider that his actions in signing the PS1 (and without identifying and responding to the issues identified by our experts), was not consistent with accepted practice at the time. In these respects, we consider that there is an applicable ground of discipline in this case, and we do not consider that any of the grounds to dismiss this matter apply. Whether Mr O'Connor's actions are a breach of his professional and ethical obligations and reach the threshold for professional discipline is a matter for a disciplinary committee.

OTHER COMMENT

110. While beyond the scope of this investigation, on the basis of the information before us, we have concerns about the development of this building. In particular, we note that a PS4 was noted as being a requirement of the building consent and that the Council requested that KOA Ltd carry out construction monitoring to a level of CM3.

111. However, there is no evidence that any construction monitoring to a level of CM3 was carried out and a PS4 was never issued. Mr O'Connor said that CM3 was declined by MTLT at the time it signed the short form contact in August 2006. Furthermore, he stated: "Please note construction observation was not required. Some observation was done post the initial engagement." However, a code compliance was issued.

112. It is unclear to us why construction monitoring to a level of CM3 was not carried out and a PS4 was not issued for this project despite them being listed as requirements in the building consent. Furthermore, if the decision was made that construction monitoring and a PS4 were not required, it is unclear how this decision was made, or whether it was simply overlooked.

CONCLUSION

113. Overall, we consider it was inappropriate for Mr O'Connor to sign the PS1 in the circumstances described above and as set out in the information considered by us above. In our opinion, Mr O'Connor should have identified the issues noted by our experts when reviewing the design for sign-off and taken additional steps to reassure himself the designs met the relevant standards before signing the PS1. We consider that this was more than a minor departure from accepted standards.

114. Accordingly, we do not consider there are grounds to reasonably dismiss the matter on the basis of the information collected to date and have decided to refer it to a disciplinary committee in accordance with rule 60(a) of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002.

Andrew McMenamin
Chair of Investigating Committee

Stewart Hobbs
Dr Sulo Shanmuganathan
Committee Members

INVESTIGATING COMMITTEE DECISION

OWN MOTION INQUIRY ABOUT KEVIN O'CONNOR — 57-65 DIXON STREET (LOT 9)

For release

In accordance with:

Chartered Professional Engineers of New Zealand Act 2002

Chartered Professional Engineers of New Zealand Rules (No 2) 2002

Issued by

Andrew McMenamin CPEng CMEngNZ

Chair of Investigating Committee

Stewart Hobbs CPEng FEngNZ IntPE (NZ)

Dr Sulo Shanmuganathan CPEng FEngNZ

Committee Members

25 March 2019

engineering new zealand

Institute of Engineering Professionals

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BACKGROUND

1. In 2015, Engineering New Zealand[1] was made aware of concerns surrounding six buildings in Masterton owned by Masterton Trust Lands Trust (MTLT). The concerns were raised by a Chartered Professional Engineer (CPEng).

2. A building located at 57-65 Dixon Street (Lot 9 DP 10491), was one of the buildings identified.

3. The concerns were about the structural integrity of the buildings. Because Engineering New Zealand has no jurisdiction over physical assets, Engineering New Zealand brought the concerns to the attention of the Ministry of Business, Innovation and Employment (MBIE), as the relevant regulatory authority.

4. MBIE subsequently commissioned GA Hughes & Associates Ltd to carry out a structural review of the buildings,[2] which identified concerns with the buildings and recommended a Detailed Seismic Assessment be carried out. Following the receipt of this report, MTLT and MBIE commissioned a Detailed Seismic Assessment to be carried out by Holmes Consulting, which identified concerns about the structural integrity of the six buildings. MTLT subsequently commenced remedial works on some of the buildings.

5. In light of the findings of these two reports, Engineering New Zealand decided that it needed to act on this information to determine if there is an issue with the engineering design of these buildings or not and, if so, what that means. It was decided that the best way to do this was by way of an own-motion inquiry pursuant to Rule 55 (1) of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002 (the Rules).

[1] On 1 October 2017 IPENZ changed its name to Engineering New Zealand. Accordingly, I will refer to Engineering New Zealand throughout this report.

[2] One of the buildings that Engineering New Zealand was initially notified about was different from the six ultimately chosen by MBIE to assess further.

SCOPE OF THE INQUIRY

6. The scope of the Inquiry was to conduct an initial investigation into:

   "The circumstances relating to the design, design review and construction monitoring of six buildings in Masterton:

   - to assess whether the Chartered Professional Engineers involved have provided engineering services in accordance with accepted standards; and
   - to learn and advise on any engineering performance and practice improvements, if necessary."

7. Kevin O'Connor & Associates Ltd (KOA Ltd) was the engineering firm involved in the engineering design of all six of the buildings. Kevin O'Connor CPEng[3], CMEngNZ[4], IntPE(NZ)[5] was identified as the Chartered Professional Engineer who signed the PS1 for the 57 – 65 Dixon Street (Lot 9) building.

8. The building was designed and built in 2010/2011.

9. The issue being considered with respect to Mr O'Connor is:

   - Whether Chartered Professional Engineer Kevin O'Connor provided engineering services relating to the engineering design work on 57-65 Dixon Street (Lot 9) in accordance with accepted standards.

INVESTIGATION

10. Following an initial investigation this matter was referred to an Investigating Committee for formal investigation. The Investigating Committee is:

    Andrew McMenamin CPEng CMEngNZ (Chair)

    Dr Sulo Shanmuganathan CPEng FEngNZ

    Stewart Hobbs CPEng FEngNZ IntPE (NZ)

11. Information considered by Engineering New Zealand for the purposes of the investigation included information from:

Kevin O'Connor	Respondent/engineer
Kevin O'Connor & Associates Ltd	Engineering company
Masterton District Council	Local authority
Masterton Trust Lands Trust	Building owner
GA Hughes & Associates Ltd	Consulting structural engineering company
Holmes Consulting	Consulting structural engineering company

[3] Chartered Professional Engineer.

[4] Engineering New Zealand Chartered Member.

[5] A member of the New Zealand section of the International Professional Engineers.

12. Independent expert advice was obtained from Barry Brown CPEng, FEngNZ[6], IntPE (NZ) (**Appendix A**) and Stuart George CPEng, IntPE (NZ), CMEngNZ (**Appendix B**).

INFORMATION GATHERED

13. Masterton Trust Lands Trust (MTLT) engaged Proarch Architects Ltd to design a new building to be used as retail space for a retailer.

14. Mr O'Connor told Engineering New Zealand that in 2010 KOA Ltd was engaged by Proarch Architects Ltd to provide the engineering design for the development. The contractual agreement was between MTLT and KOA Ltd. However, MTLT told Engineering New Zealand that Proarch Architects Ltd acted as the project manager and all communications with KOA Ltd and the Masterton District Council went through Proarch Architects Ltd.

15. The scope of KOA Ltd's engagement included site soil testing, design and detailing of the proposed building, site work plans and services, and construction monitoring to a CM3 level.[7]

BUILDING DESIGN

16. The building is a single storey 'L' shaped building with a mezzanine floor, constructed with steel portal frames and concrete precast panels. The offices on the mezzanine floor are partitions constructed from timber. The floor is concrete slab on grade.

STRUCTURAL CONCEPT

17. The roof is supported by steel purlins and precast concrete walls. The steel purlins are supported by steel UB portal frames.

18. The concrete panels are supported at the base by slab thickenings.

19. The lateral wind and earthquake loads are transferred by steel PFC transoms to the UB portal frame. Loads within the portal frames are transferred into the concrete piles and then into the surrounding soils.

ENGINEERING DESIGN WORK

20. The engineering designs held on the Masterton District Council property file (sheets S1- S16, SD1, SD2, SW1-5 (Rev A)) are dated as being issued for building consent on 2 October 2010. A transfer sheet held on the property file shows that these plans were sent to Proarch Architects Ltd on 2 November 2010, together with the calculations and specifications.

21. Mr O'Connor told Engineering New Zealand that the engineer who completed the designs was KOA Ltd employee, Mr B (MIPENZ at the time of these events), under the supervision of a senior engineer. Mr O'Connor did not advise the name of the senior engineer. However, he said that it would have been one of two senior engineers in the KOA Ltd office, both of whom were very experienced.

[6] Fellow of Engineering New Zealand.

[7] CM3 monitoring is defined as: "Review, to an extent agreed with the client, random samples of important work procedures, for compliance with the requirements of the plans and specifications and review important completed work prior to enclosure or on completion as appropriate."

22. Mr O'Connor said that his role was to "set up the job to determine who would do the design work". He said that he also acted as the client liaison. On the plans it is listed that Mr B was the designer, that they were "drawn" by "TRG/JCR"[8] and "checked" by "proarch".

23. Mr O'Connor told Engineering New Zealand that Mr B developed the designs using Microstran structural analysis software, "using the equivalent static method of analysis and application of codes". Mr O'Connor said that from the plans it appears that Mr B used a ductility factor (μ) of 1.25 and that the "KOA records show [Mr B] used the following applicable design codes: NZS1170, NZS3604, NZS3404, NZS3606, NZS3101".

24. In a letter dated 17 November 2010 from Spencer Holmes Ltd to Masterton District Council relating to a 'screening review' it had completed (discussed further below), reference is made to a PS1 dated 28 October 2010 that was submitted as part of the original building consent application. However, this PS1 is not on the Masterton District Council property file or on the MTLT records. Neither Mr O'Connor nor Proarch Architects have provided a copy of this PS1, despite requests to do so.

25. Mr O'Connor said that, in terms of quality assurance , "the plans would have been subject to a normal dimensional check and a structural check by the designer and in this case the designer's supervisor before being issued". Furthermore, Mr O'Connor told the Investigating Committee that he had overall responsibility for the designs, and before signing them off he would have carried out a quick high level check of the designs and calculations but that he relied on the review carried out by the senior engineer. Mr O'Connor said there was no way of confirming the senior engineer had completed their check, or the extent of that check, and would take the engineer's word the check had been completed.

BUILDING CONSENT

26. On 3 November 2010, a building consent application was submitted to Masterton District Council.

27. On 8 November 2010, Spencer Holmes Ltd was engaged by Masterton District Council to carry out a "screening review".

28. On 17 November 2010, Spencer Holmes Ltd wrote to Masterton District Council advising that "Spencer Holmes Ltd has undertaken a screening of this application for B1 structure outside the scope of the Accepted Solutions of the Building Consent Handbook of the Building Code".

29. Spencer Holmes Ltd noted that KOA Ltd's Producer Statement dated 28 October 2010, the architectural drawings, the engineering drawings, and calculations and specifications had been supplied and that:

> "[w]e consider that these documents provide reasonable grounds for Masterton District Council to accept a producer statement PS1 as a basis to issue Building Consent subject to;
>
> The designer or other approved engineer;
>
> - Verifying that the ultimate ground bearing capacity and foundation conditions are in accordance with design assumptions of 300 kPa ultimate bearing capacity; and

[8] We do not know who TRG or JCR are.

- Construction monitoring of the structural aspects to ACENZ/IPENZ level CM3 and provision of a Producer Statement PS4 – Construction Review for the construction."

30. There is no further correspondence on the property file in relation to this review. It is therefore unclear if these recommendations from Spencer Holmes Ltd were ever communicated to any other party or actioned. When asked by Engineering New Zealand for any correspondence in relation to this review Mr O'Connor stated: "I do not know who reviewed the building for the Masterton District Council. There was no correspondence from the council or the council reviewing engineer to KOA, raising any issues."

31. On 24 November 2010, Masterton District Council wrote to Proarch Architects Ltd requesting additional information including, "The producer statement from Kevin O'Connor needs to reference all the plans & other documents it covers."

32. Mr O'Connor was asked about the reason for this request. In response Mr O'Connor stated: "No correspondence exists on KOA files in respect of this. … I can only assume this was very much part of the normal review process with council where we responded to a request."

33. However, on 25 November 2010, a PS1 was signed by Kevin O'Connor which lists all the proposed building work covered by the document. In the PS1 Mr O'Connor certified that:

    "subject to:

    (i) The site verification of the following design assumptions – safe ground bearing capacity in excess of 100 kPa, and
    (ii) All proprietary products meeting the performance specification requirements….

    … the drawings, specifications, and other documents provide or listed in the attached schedule according to which it is proposed that the building be constructed, comply with the relevant provisions of the Building Code."

34. On 30 November 2010, building consent was granted.

CONSTRUCTION

35. Construction commenced in 2010. During construction an amendment to the initial design for "Alteration to Internal Partition Layouts" was made. The revised engineering calculations are dated December 2010 and initialled by Mr B.

36. Construction was completed mid-2011.

37. A producer statement – PS4 – construction (PS4) was signed by Kevin O'Connor. It is undated.

38. On 1 March 2011, it is noted in the Masterton District Council records that "Percolations tests required as per plan documents" prior to a Code Compliance Certificate being granted. These were received by Masterton District Council on 12 April 2011.

39. On 4 August 2011, a compliance schedule was issued.

40. On 7 November 2011, a Code Compliance Certificate was granted.

SUBSEQUENT REVIEWS OF DESIGNS

41. On 4 August 2014, MTLT wrote to Proarch Architects Ltd, noting that it had become aware of engineering design issues relating to another project that KOA Ltd had provided engineering design for. MTLT requested advice as to whether similar issues may also apply to any of its buildings that KOA Ltd had been involved in. This letter was forwarded to KOA Ltd. On 23 September 2014, Mr O'Connor responded directly to MTLT stating that he had carried out a desktop review of all the relevant buildings.[9] He stated that following his review, in his opinion, "the buildings … [have] been properly designed and detailed".

42. However, he stated:

> "In terms of the [..]building at 57-65 (Lot 9) Dixon Street, Masterton I believe there is a minor issue with the roof bracing which requires the addition of a strut, we would like to take a look at the building and detail any necessary modification."[10]

43. On 3 November 2014, MTLT responded that a review by Mr O'Connor or any current KOA Ltd employee did not provide the level of necessary comfort from an independent perspective. MTLT requested advice on how this could be achieved and whether it should consider requesting a peer review.

44. On 21 November 2014, Mr O'Connor responded stating that "I fully understand that KOA could be seen to have a financial interest which could be seen as impairing the personal judgement of the reviewers" and recommended that MTLT seek a detailed seismic assessment.

45. It does not appear that any further action was taken by MTLT at that time.

GA HUGHES & ASSOCIATES LTD

46. As noted above, in June 2016, GA Hughes & Associates Ltd were engaged by MBIE to carry out a "High Level Structural Review" of the building. The information considered as part of the Hughes review included the Masterton District Council property file, including the engineering calculations and drawings. A site visit and visual inspection of the building was also carried out. GA Hughes & Associates Ltd also undertook an Initial Evaluation Procedure (IEP) for the building, which is a quick assessment method based on factors such as the buildings age, the type of materials used, and the construction type, used to identify if a building is potentially earthquake prone.

47. The review identified the following structural vulnerabilities:

> "The roof bracing and load paths are not clear. Additional bracing is required between different levels of the roof. There is no consideration of the Cs factor in the bracing design.
>
> The transom steel members supporting lateral loads from the concrete wall panels should be reviewed to confirm they are adequate under the parts provision of the loading standard.

[9] This included 57-65 Dixon Street (Lot 9).

[10] It is unclear whether Mr O'Connor ultimately looked at the building.

The base fixings of the concrete wall panels using Reid inserts which will be in the cracked zone for opening actions. This will reduce capacity of the inserts. There are concerns about the capacity of the pile assumed to provide fixity at the portal legs.

Fly bracing has been used to provide rafter restraint but appears to have been omitted in several locations and restraint of portal legs should be reviewed."

HOLMES CONSULTING

48. In September 2016, Holmes Consulting were engaged by MBIE to carry out a "Detailed Seismic Assessment with review of the structural capacity of vulnerabilities … as identified by previous reporting by G. A. Hughes".

49. The report noted that the "principal elements of our scope are to provide an independent review of load paths and configuration, as well as key detailing that may affect the seismic capacity …". The assessment involved review of documentation and calculations, where necessary. The assessment used the following approach:

    "Assess the building performance using the 2006 Assessment and Improvement of the Structural Performance of Buildings in Earthquakes (Red Book) and the draft for upcoming Seismic Assessment of Existing Buildings (Part C6)."

50. The review estimated the building's strength to be 55-65% of the New Building Standard (NBS). It stated:

    "This result comes from the out-of-plane wall cantilever capacity. It is noted however that the roof bracing providing out-of-plane ties to the walls will reach their ultimate capacity at 30-35% of current code demand, however the precast walls have a base connection capable of sustaining out-of-plane moments, although it is possible that the foundations will rock at a similar level to the moment demands."

51. In relation to the metal strap bracing as shown on the plans, it noted that "it is likely this bracing will yield under moderate earthquake loads, rendering it ineffective as a lateral load path". It is noted that therefore the primary lateral load system is the out-of-plane bending of the precast concrete walls.

52. The assessment considered the seismic loads being applied in any direction and advised:

    "precast concrete walls within the building have been analysed as nominally ductile with a strength reduction factor of 0.7. As a result, the 185 millimetre thick wall along the building's east elevation has an estimated strength of 60% of the strength of an equivalent new building."

53. It noted that it is difficult to predict the behaviour of singly reinforced concrete walls under biaxial bending. It advised that in this building "walls subjected to biaxial bending would crack on an inclined plane through the wall's thickness. As a result, when lateral loads on the wall reverse, and cracks formed under tension close, the potential for the wall becoming offset across an inclined crack and bar buckling could lead to wall instability. Potentially, this would compromise a critical vertical load path for the building and severely limit the concrete wall's ability to resist out-of-plane loads". As a result, a ductility factor higher than $\mu = 1.25$ is not considered applicable in the assessment of the seismic capacity.

EXPERT ADVICE

54. Engineering New Zealand engaged two independent expert structural engineers — Stuart George and Barry Brown — to provide advice on this matter. They were asked to advise whether, in their opinion, the engineering designs were produced in accordance with accepted standards at the time and consistent with what a reasonable engineer, in the same circumstances, would likely have done. They were not asked to undertake a seismic assessment of the building and neither have undertaken an evaluation of the seismic performance rating. This is a key difference in the focus of the advice provided by Mr George and Mr Brown compared to the assessments undertake by GA Hughes & Associates Ltd and Holmes Consulting – the Hughes and Holmes reports are instead concerned with the building's current performance.

55. The information Mr Brown and Mr George considered in forming their opinions included the Masterton District Council property file, Mr O'Connor's submissions, as well as the GA Hughes & Associates Ltd and Holmes Consulting reports.

56. In assessing whether the overall design met acceptable standards Mr George stated:

> "The definition of meeting acceptable standards is difficult to define. My best analogy would be marking a project at Engineering School, it is inevitable that [there] will be a wide variation in marks, and unlikely that anyone would score 100%. Even with the benefit of years of experience and an engineering registration process engineering knowledge and judgement varies."

57. Mr George gave the design an overall grade "C+", noting the "[b]racing load path at Grid B [is] insufficiently detailed". He also identified concerns with the connection to the existing building.

58. Mr Brown's primary concern related to the wall panel/foundation connection. He stated:

> "Most serious primary defect is the strength deficit within the 6 m high wall panel/foundation connection … when subject to face loads under ULS design conditions, with this deficiency exacerbated when the load effects of in-plane shears are added."

59. Mr Brown also identified secondary defects relating to the canopy rafter connection to the column, as well as the transom/wall panel connection.

60. See **Appendix A** and **B** for a full copy of their reports.

RESPONSE FROM MR O'CONNOR

61. In a statement to Engineering New Zealand dated 29 May 2017 Mr O'Connor stated that, in his opinion, the engineering services provided were in accordance with accepted design standards. He stated: "There is no question of public safety here." Mr O'Connor said that he considers that the building is "compliant", with the exception of a "potential roof bracing issue which was identified and the client advised". Mr O'Connor stated:

> "The original calculations are correct within themselves in that the loads are calculated and members accordingly sized, however the original designer may have miscalculated

(underestimated) the earthquake coefficient, but overestimated the loads for the earthquake load case."

62. Mr O'Connor stated that a review carried out by a senior KOA Ltd engineer "using an equivalent static analysis advised that the building whilst being a code minimum building it nonetheless was complaint".[11] Mr O'Connor advised that KOA Ltd has also rechecked a number of the design elements which Mr O'Connor says demonstrates the building's compliance. Mr O'Connor has provided copies of his calculations.

63. Furthermore, Mr O'Connor stated that following a detailed review using a "3-Dimensional Non-Linear Push-over Analysis (NLPA) using a ductility factor (μ) of 1.25, "a further strut may be required along line B along with [cross] bracing in this wall".

Comment on the GA Hughes & Associates Ltd report

64. In relation to the GA Hughes & Associates Ltd report Mr O'Connor noted that this review was an Initial Evaluation Procedure (IEP) and based on a document review and visual building inspection only and "cannot be relied on to be accurate".

65. In relation to the comments in the report that the transoms and panel fixings needed a detailed assessment with respect to the parts provision of the design standards Mr O'Connor stated that the transoms and panel fixings panels were designed to the relevant code. He stated:

   "…it was common amongst engineers to not design precast panels supported on the ground to the parts provision."

66. In relation to the rafter and leg restraint, Mr O'Connor said that a ductility factor (μ) of 1.25 had been used and that the earthquake coefficients had been underestimated but that the structure's weight had been overestimated with the resultant effect being "the portals would in general be adequate except for the column base capacity". Furthermore. Mr O'Connor said that the NLPA "shows these are adequate".

67. In relation to the roof bracing Mr O'Connor stated: "We have identified the possible need for some additional bracing/struts."

68. In relation to the adequacy of the foundations Mr O'Connor stated:

   "[T]he concrete bored piles are founded in river gravels and photographs taken during site inspections show the pile holes augered are in excess of the 600 mm [diameter] specified. Regardless the pile capacity at 600mm diameter in this soil is at least 50% greater than the demand and far greater than the capacity of the member base moment capacity. Additionally, the steel portal base fixity is a MEPS 50/25 connection with a capacity equal to the equivalent static demand load. Thus the foundations cannot be overloaded."

69. In relation to the reference to the building being irregular in shape, Mr O'Connor stated that it is not irregular. He noted that while it may be considered irregular in stiffness "with a centre of rigidity not coincident with the centre of mass", he stated "this is not an issue as these elements are connected via

[11] Equivalent Static Analysis is a method used to analyse the seismic performance of a building.

a flexible diaphragm" which "allows a 30% reduction in capacity" resulting in a % NBS of 79% and 100% in each direction, rather than 55% and 75% respectively.

Comment on the Holmes Consulting report

70. In relation to the Holmes Consulting report Mr O'Connor requested that the report's limitations be noted in that the assessment was based on review of the drawings and calculations only.

71. In relation to the opinion that additional transoms are required to connect to those existing, Mr O'Connor stated: "I am unsure why this comment has been made as our analysis shows the transoms to be adequate."

72. In relation to the advice that additional fly-bracing, as well as connections between the roof panels and precast concrete walls on the east and west walls is needed, Mr O'Connor again stated that KOA Ltd analysis has shown that both these elements are "adequate".

73. In relation to the advice that the portals require strengthening to reduce drifts, Mr O'Connor noted that "there is no specific limit to drift in NZS 1170 or NZS 3404" and the 2.5% limit referred to relates to "an inter-storey deflection level" which Mr O'Connor advised does not apply in this case. Furthermore, Mr O'Connor stated that from their analysis "the drift in the direction of the portals is a maximum of 100mm (1.4%)".

74. In relation to the bracing connections, Mr O'Connor stated: "KOA have shown that in conducting an analysis of [57-65 Dixon Street (Lot 9)and 57-65 Dixon Street building as a joint structure … that some additional bracing may be required …".

General comment

75. Overall, in relation to both the reports Mr O'Connor stated:

> "The reports question the adequacy of a) eccentric transom connections and b) panel connections to foundations without presenting any justification or evidence for these views. Our review and observations show these details are adequate and further common details found in many buildings designed by many other engineers."

RESPONSE TO PROVISIONAL DECISION

76. In response to the provisional decision Mr O'Connor noted that in its decision the Investigating Committee is considering his actions in relation to the signing of the PS1, but has not identified what accepted standards in relation to the use of PS1s are. Mr O'Connor refers to Engineering New Zealand's Practice Note 1 Guidelines on Producer Statements,[12] noting it states that "limited standardised practice has developed" in relation to the use of PS1s and that a producer statement is "not a product warranty or guarantee of compliance".

77. Further to this, Mr O'Connor reiterated his view that, based on the information he had available when signing the PS1, he had reasonable grounds to believe that the designs complied with the Building Code. Mr O'Connor stated:

[12] See: https://www.engineeringnz.org/resources/practice-notes-and-guidelines/

"In signing the PS1 I confirmed that I had a reasonable basis for thinking that, in my opinion, the designs complied with the Building Code. I did have a reasonable basis for thinking the designs complied with the Building Code. … [I]t was my practice to check aspects of the calculations in the designs, and to place weight on the fact the designs had been prepared and/or reviewed by an engineer with suitable experience and the person doing this work had 20 years experience as an engineer and the building is a relatively low risk structure smaller than most houses."

78. Mr O'Connor noted that there was no requirement for him to "to exhaustively review the designs and calculations for the purpose of preparing the PS1".

79. In addition, Mr O'Connor expressed concern regarding the conclusion in Mr Brown's advice report of 30 April 2018, in which Mr Brown stated: "if the output PS1 for design … signed off by the CPEng/Principal is defective in terms of its statement … the author must accept responsibility arising from that." In relation to this statement Mr O'Connor stated:

> "This is not the inevitable consequence of a defective design, if the author of the producer statement took reasonable steps before signing it. A producer statement states that the author believes on reasonable grounds etc… Thus the issue is whether reasonable steps were taken, but in hindsight if those steps are found to reveal an error may have occurred that does not make the process unreasonable."

80. Mr O'Connor stated that the environment and standards have changed significantly since the Canterbury earthquakes. Mr O'Connor stated:

> "The environment we work in now as engineers and the standards expected have significantly changed because of the scrutiny applied to the profession since the Christchurch earthquakes. I believe if such scrutiny was applied to engineering work prior to this time, many practices and design details when judged by today's standards, would be discovered and deemed deficient."

81. Mr O'Connor reiterated his submission that many of the details that have been identified as being deficient in these cases were commonplace at the time the buildings were designed.

DISCUSSION

INVESTIGATING COMMITTEE'S ROLE AND THE LEGAL CONTEXT

82. Professional disciplinary processes primarily exist to protect the public, uphold professional standards, and maintain public confidence in the profession and its regulation. They do this by ensuring that members of the profession adhere to certain universal (or accepted) professional standards.[13]

83. Our role in this professional disciplinary process is to determine whether there are grounds to dismiss the matter as set out in rule 57 of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002 (the Rules). If none of these grounds to dismiss the matter apply, then the matter must be referred to a Disciplinary Committee in accordance with rule 60(a) of the Rules.

[13] *Dentice v Valuers Registration Board* [1992] 1 NZLR 720 (HC).

84. In order to determine whether there are grounds to dismiss the matter, we have investigated whether Mr O'Connor provided engineering services related to the engineering design work on the building at 57-65 Dixon Street (Lot 9)in accordance with accepted professional standards at the time the building was designed. We are not concerned with whether the building strictly complied with the building code or the building's current seismic rating – our assessment is whether Mr O'Connor acted reasonably at all stages of his involvement in the design of this building, including when he reviewed the design and signed the PS1.

85. The legal test we need to assess Mr O'Connor's actions against is whether he acted reasonably and in accordance with what a reasonable body of his peers would have done in the same situation (i.e. did he act in accordance with accepted standards).

86. If the evidence is that Mr O'Connor acted in accordance with accepted standards then we can dismiss the matter.[14] If the evidence is that Mr O'Connor did not act in accordance with accepted standards then we need to assess how significant his departure from accepted standards was – if it was minor, we may dismiss the matter as insufficiently grave to warrant further investigation;[15] if it is more than minor, and no other ground for dismissal applies,[16] we are required to refer the matter to a disciplinary committee.

87. The issue we have specifically considered in this case is whether in signing the PS1 Mr O'Connor was acting in accordance with accepted professional standards at the time the building was designed.

REVIEW AND SIGN OFF OF THE DESIGN BY WAY OF PS1

Adequacy of the design

88. Both Mr George and Mr Brown identified concerns with the engineering design of this building, primarily relating to the foundation/wall connections.

89. Mr Brown advised that his calculations suggest there is a "[p]otentially significant deficit in [the] wall/foundation (TCM) joint". Mr George considered that the "[b]racing load path at Grid B to be insufficiently detailed. He also raised concern about the connection to the existing building.

90. Mr O'Connor submitted that the building at 57-65 Dixon Street (Lot 9) is "compliant", with the exception of a "potential roof bracing issue which was identified and the client advised", but that there is no issue of public safety, and that this has been confirmed through reviews using different methods.

91. Further to this, in response to the provisional opinion, Mr O'Connor submitted that many of the details that have been identified as being deficient were commonplace at the time of the building's design and are the same as those used by Holmes Consulting in recent examples. Mr O'Connor stated:

[14] Rule 57(a) of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002, on the basis that there is no applicable ground of discipline under section 21(1)(a) to (d) of the Act.

[15] Rule 57(ba) of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002.

[16] There are other grounds in the legislation for dismissing a complaint, including: where the subject matter of the complaint is trivial; the complaint is frivolous or vexatious or is not made in good faith; where the person alleged to be aggrieved does not wish action to be taken or continued; where the complainant doesn't have a sufficient personal interest in the subject matter of the complaint; or an investigation is no longer practicable or desirable given the time elapsed since the matter giving rise to the complaint (Rule 57 of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002. In our view, given the nature of the Inquiry, we cannot apply any of these grounds to reasonably dismiss this matter.

"The environment we work in now as engineers and the standards expected have significantly changed because of the scrutiny applied to the profession since the Christchurch earthquakes. I believe if such scrutiny was applied to engineering work prior to this time, many practices and design details when judged by today's standards, would be discovered and deemed deficient."

92. As noted above, our experts were asked to advise whether, in their opinion, the engineering designs were produced in accordance with accepted standards at the time and consistent with what a reasonable engineer, in the same circumstances, would likely have done. While the GA Hughes & Associates Ltd and Holmes Consulting reports focus on the current performance of the buildings and the issues identified relate to current standards, Mr George and Mr Brown's advice relate to the accepted standards at the time the buildings were designed – 2010 in the case of the building at 57-65 Dixon Street (Lot 9).

93. We note that both the experts considered all the information provided to Engineering New Zealand by Mr O'Connor, including his submission of 29 May 2017, as well as a summary of the Investigating Committee's interview with Mr O'Connor on 26 March 2018.

94. Mr O'Connor has provided us with no new information. Accordingly, guided by our experts, we confirm our concern that there are deficiencies with the original engineering design of this building that indicate it was not of an adequate standard, taking into account the year that it was designed.

Should Mr O'Connor have signed the PS1?

95. Chartered professional engineers use PS1s to confirm their professional opinion that aspects of a building's design comply with the Building Code. Their intent is to signal to a building consent authority (BCA) that certain design work has been done (or overseen/supervised) by a practitioner who is competent to perform the defined work.[17]

96. Although they have no legal status under the Building Act 2004, PS1s are intended to provide BCAs with information to establish that there are reasonable grounds for the issue of a building consent. When used properly, they give BCAs confidence that certain building work will be constructed to meet the Building Code.

97. Chartered professional engineers should be aware that BCAs are likely to rely on producer statements to some extent, and therefore should be mindful that responsibilities and potential liabilities may arise from signing them.[18]

98. In signing the PS1 for the building at 57-65 Dixon Street (Lot 9) Mr O'Connor was confirming his professional opinion that aspects of the building's design complied with the Building Code. Accordingly, by signing the PS1 Mr O'Connor assumed responsibility for the building's design.

99. Therefore, the question for us to consider is whether it was reasonable for Mr O'Connor to sign off the building's design by way of PS1, or whether he should have identified and responded to those issues identified by our experts, as noted above. In other words, the question for us is whether Mr O'Connor carried out his part in the review and sign out process in a careful and competent manner and in accordance with accepted professional standards.

[17] Engineering New Zealand Practice Note 1 *Guidelines on Producer Statements* (January 2014), section 3.1.

[18] Ibid.

100. The starting point for this assessment is to consider Mr O'Connor's role in the review and sign out process.

101. In his statement dated 29 May 2017, Mr O'Connor told Engineering New Zealand that his role in the design and development of the building at 57-65 Dixon Street (Lot 9) was limited to acting as the client interface and providing the final review and sign-off of the design by way of PS1.

102. Mr O'Connor told the Investigating Committee during an interview that in signing off the building he would have carried out a quick check of the calculations and a high level review of the designs, but said that he was reliant on the senior engineer carrying out their structural check of the designs. Mr O'Connor said that the senior engineer involved in this project would have been one of two senior engineers in the office, both of whom had a significant amount of experience. However, he said that he had no way of confirming the check had been completed or to what extent - he was reliant on the engineer's word.

103. Mr O'Connor noted in his response to the provisional decision that there is no standard practice in relation to the use of PS1s and he refers to Engineering New Zealand's Practice Note 1 *Guidelines on Producer Statements* (the Practice Note) to support this. He also said that there was no requirement for him to "exhaustively review the designs and calculations for the purposes of preparing the PS1". Mr O'Connor submitted that based on his review and assurance provided by the senior engineer involved he had "a reasonable basis for thinking the designs complied with the Building Code".

104. The part of the Practice Note that Mr O'Connor refers to in support of his argument about standard practice states: "By the same token there is no provision for BCAs to require a producer statement, as of right. As a result of their non-mandatory status, limited standardised practice has developed. There is currently such a wide variation in the way BCAs accept and/or rely on producer statements that there is a resultant degree of confusion, frustration and inefficiency amongst practitioners and BCAs." We interpret the Practice Note to be saying that the lack of standardised practice related to the extent by which producer statements are used by BCAs; not the extent of engineering practice with regard to the level of review a chartered professional engineer should carry out before signing off a PS1 – which is the issue in this case. We consider that, in 2010, there would be a clear expectation from Mr O'Connor's peers that a chartered professional engineer signing off a design by way of PS1 would take sufficient steps in their review of a design to satisfy themselves that they had reasonable grounds for signing the PS1.

105. We accept there was no requirement to undertake an "exhaustive" review himself. However, as noted above, in signing the PS1 Mr O'Connor was confirming his professional opinion that aspects of the building's design complied with the Building Code. Regardless of Mr O'Connor's trust in the experience and competence of the designer or the senior engineer who carried out the checks, as a CPEng engineer and the person signing out the designs, Mr O'Connor had a responsibility to carry out an adequate review himself, and not just rely on the reviews that may or may not have been carried out at other steps of the system. As noted by Mr Brown:[19]

> "if the output PS1 for design … signed off by the CPEng/Principal is defective in terms of its statements (and irrespective of the reliance placed on verbal advice given by a Senior Engineer), the author must accept the responsibility arising from that. Put another way, the process that the

[19] See report dated 30 April 2018.

CPEng/Principal used in reaching his/her judgement regarding the sufficiency of building design documentation is irrelevant if the statement is defective."

106. We note that in response to the provisional decision Mr O'Connor argued that if the author of a PS1 took reasonable steps to check the design met the relevant standards, if the error is later identified, "that does not make the process unreasonable".

107. We agree with Mr O'Connor that the issue is whether he took reasonable steps to check the designs.

108. Mr George considered the issues he identified should have been picked up during the design and checking stages of the design development. There is no evidence that they were.

109. Overall, when weighing up the advice from the experts, it is our opinion that Mr O'Connor did not act in accordance with accepted standards in signing off this design by way of PS1. We consider that, as the CPEng engineer signing off the design, Mr O'Connor should have identified the issues as noted by our experts and, in the very least, taken additional steps to in light of those issues to satisfy himself that the building met relevant standards before signing the PS1. There is no evidence that he did.

110. Further, in our view, a reasonable engineer would document any process of checking and clarifying aspects of a design at the sign-out stage and included documentation to support their assertions.

111. In our opinion, a reasonable body of Mr O'Connor's peers would likely consider that his actions in signing the PS1 (and without identifying and responding to the issues identified by our experts), was not consistent with accepted practice at the time. In these respects, we consider that there is an applicable ground of discipline in this case, and we do not consider that any of the grounds to dismiss this matter apply. Whether Mr O'Connor's actions are a breach of his professional and ethical obligations and reach the threshold for professional discipline is a matter for a disciplinary committee.

112. We note that Mr O'Connor told the Investigating Committee that in hindsight, he should not have signed the PS1 at the time and that the building could have benefitted from some additional fly bracing to provide additional support.

CONCLUSION

113. Overall, we consider it was inappropriate for Mr O'Connor to sign the PS1 in the circumstances described above and as set out in the information considered by us above. In our opinion, Mr O'Connor should have identified the issues noted by our experts when reviewing the design for sign-off and taken additional steps to reassure himself the designs met the relevant standards before signing the PS1. We consider this was more than a minor departure from accepted standards.

114. Accordingly, we do not consider that there are grounds to reasonably dismiss the matter on the basis of the information collected to date and have decided to refer it to a disciplinary committee in accordance with rule 60(a) of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002.

Andrew McMenamin CPEng CMEng
Chair of Investigating Committee

Stewart Hobbs CPEng FEngNZ IntPE(NZ)
Dr Sulo Shanmuganathan CPEng FEngNZ
Committee Members

INVESTIGATING COMMITTEE DECISION

OWN MOTION INQUIRY ABOUT KEVIN O'CONNOR — 96-120 QUEEN STREET

For release

In accordance with:

Chartered Professional Engineers of New Zealand Act 2002
Chartered Professional Engineers of New Zealand Rules (No 2) 2002

Issued by

Andrew McMenamin CPEng CMEngNZ
Chair of Investigating Committee

Stewart Hobbs CPEng FEngNZ IntPE(NZ)
Dr Sulo Shanmuganathan CPEng FEngNZ
Committee Members

24 March 2019

engineering new zealand
Institute of Engineering Professionals

CONTENTS

BACKGROUND

1. In 2015, Engineering New Zealand[1] was made aware of concerns surrounding six buildings in Masterton owned by Masterton Trust Lands Trust (MTLT). The concerns were raised by a Chartered Professional Engineer (CPEng).

2. A building at 96-120 Queen Street, was one of the buildings identified.

3. The concerns were about the structural integrity of the buildings. Because Engineering New Zealand has no jurisdiction over physical assets, Engineering New Zealand brought the concerns to the attention of the Ministry of Business, Innovation and Employment (MBIE), as the relevant regulatory authority.

4. MBIE subsequently commissioned GA Hughes & Associates Ltd to carry out a structural review of the buildings,[2] which identified concerns with the buildings and recommended a Detailed Seismic Assessment be carried out. Following the receipt of this report, MTLT and MBIE commissioned a Detailed Seismic Assessment to be carried out by Holmes Consulting, which identified concerns about the structural integrity of the six buildings. MTLT subsequently commenced remedial works on some of the buildings.

5. In light of the findings of these two reports Engineering New Zealand decided that it needed to act on this information to determine if there is an issue with the engineering design of these buildings or not and, if so, what that means. It was decided that the best way to do this was by way of an own-motion inquiry pursuant to Rule 55 (1) of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002 (the Rules).

[1] On 1 October 2017 IPENZ changed its name to Engineering New Zealand. Accordingly, I will refer to Engineering New Zealand throughout this report.

[2] One of the buildings that Engineering New Zealand was initially notified about was different from the six ultimately chosen by MBIE to assess further.

SCOPE OF INQUIRY

6. The scope of the Inquiry was to conduct an initial investigation into:

   "The circumstances relating to the design, design review and construction monitoring of six buildings in Masterton:

   - to assess whether the Chartered Professional Engineers involved have provided engineering services in accordance with accepted standards; and
   - to learn and advise on any engineering performance and practice improvements, if necessary."

7. Kevin O'Connor & Associates (KOA Ltd) was the engineering firm involved in the engineering design of all six of the buildings. Kevin O'Connor CPEng CMEngNZ[3]IntPE(NZ)[4] was identified as the Chartered Professional Engineer who signed the PS1 for the building at 96-120 Queen Street.

8. The building at 96-120 Queen Street designed and built in 2011.

9. The issue being considered with respect to Mr O'Connor is:

- Whether Chartered Professional Engineer Kevin O'Connor provided engineering services relating to the engineering design work on 96-120 Queen Street in accordance with accepted standards.

INVESTIGATION

10. Following an initial investigation this matter was referred to an Investigating Committee for formal investigation. The Investigating Committee is:

    Andrew McMenamin CPEng CMEngNZ (Chair)

    Dr Sulo Shanmuganathan CPEng FEngNZ

    Stewart Hobbs CPEng FEngNZ IntPE (NZ)

11. Information collected by Engineering New Zealand for the purposes of the investigation included information from:

Kevin O'Connor	Respondent/engineer
Kevin O'Connor & Associates Ltd	Engineering company
Masterton District Council	Local authority
Masterton Trust Lands Trust	Building owner
GA Hughes & Associates Ltd	Consulting structural engineering company
Holmes Consulting	Consulting structural engineering company

12. Independent expert advice was obtained from Barry Brown CPEng FEngNZ[5] IntPE(NZ) (**Appendix A**) and Stuart George CPEng CMEngNZ IntPE(NZ) (**Appendix B**).

[3] Engineering New Zealand Chartered Member.

[4] A member of the New Zealand section of the International Professional Engineers.

[5] Engineering New Zealand Fellow.

INFORMATION GATHERED

13. Masterton Trust Lands Trust (MTLT) engaged Proarch Architects Ltd to design a new building to be used as a cycle shop - retail and workshop, located at 96-120 Queen Street (Lot 1, DP 441493).

14. MTLT also engaged Kevin O'Connor & Associates Ltd (KOA Ltd) to provide the engineering design for the development. While the contractual engagement was between MTLT and KOA Ltd, MTLT told Engineering New Zealand that Proarch Architects Ltd acted as the project manager and all communications with KOA Ltd and the Masterton District Council went through Proarch Architects Ltd. The Short Form Agreement for Consultant Engagement signed and dated by Mr O'Connor on 11 April 2011 states that the agreement was between "Masterton Trust Lands Trust C/- Proarch Architects Ltd (*client)*" (sic) and "Kevin O'Connor & Associates Ltd (*Consultant)*". The letter of acceptance from Kevin O'Connor is addressed to "Masterton Trust Lands Trust C/- Proarch Architects Ltd" (sic).

15. The scope of KOA Ltd's engagement included structural design of the proposed building, structural calculations and answering questions from the Council and client liaison, the provision of CAD[6] drawings and specifications, and the provision of a Producer Statement – PS1 – Design. It also included construction monitoring to a CM3 level.[7]

BUILDING DESIGN

16. The building is a two-level commercial/retail rectangular building, constructed predominantly with steel portal frames and concrete precast panel walls. The floor is 150mm reinforced concrete slab on grade, with regularly spaced piled foundations, to 200mm thick. The first floor is structural steel supporting a timber framed floor.

ENGINEERING DESIGN

17. The engineering designs are dated 22 June 2011 and recorded as being "issued for consent only". The calculations are initialled by E.L and the designs are initialled as being designed by two different engineers, drawn by a third engineer, and checked by the design engineers. Mr O'Connor told Engineering New Zealand that Mr B, an employee of KOA Ltd working out of the Palmerston North KOA Ltd office was one of the design engineers, and that Mr B completed the designs under the supervision of a senior, non-CPEng engineer. Mr O'Connor did not specify who the senior engineer was.

18. Mr O'Connor told Engineering New Zealand that Mr B used Microstran structural software for the "overall building analysis, using the equivalent static method analysis and applicable design codes; NZS 1170, NZS3101, NZS 3404 and NZS3603". A ductility factor of 3 was used.

19. In relation to the quality assurance processes, Mr O'Connor stated: "…the plans would have been subject to a normal dimensional check and a structural check by the designer and in this case the designer's supervisor before being issued."

20. There is no design features report for the design.

[6] A type of design software.

[7] CM3 monitoring is defined as: "Review, to an extent agreed with the client, random samples of important work procedures, for compliance with the requirements of the plans and specifications and review important completed work prior to enclosure or on completion as appropriate."

STRUCTURAL CONCEPT

21. Steel UB portal frames, spaced at 2.4 metres, stabilise two levels of 150mm precast concrete wall panels vertically on the eastern elevation. At the western end, the rafters are supported by 360UB45 columns. Several partial-height 150 mm thick precast panels are located intermittently along the building's west and south elevations.

22. According to the GA Hughes & Associates Ltd report, lateral wind and earthquake loads across the building are resisted by fixity at the base of the portal leg. Wall loads at the roof are transferred to the portal rafter by a 300PFC transom. Fire face loads are provided for by cantilevered concrete panels above the foundation.

BUILDING CONSENT

23. On 21 June 2011, a Producer Statement — PS1 — Design (PS1) was issued by KOA Ltd. This was signed by Kevin O'Connor. In the PS1 Mr O'Connor certified that he believed "[o]n behalf of the **Design Firm**, and subject to:

    (i) The site verification of the following design assumptions – safe ground bearing capacity in excess of 100kPa; and

    (ii) All proprietary products meeting the performance specification requirements.

    the drawings, specifications, and other documents provided or listed in the attached schedule according to which it is proposed that the building be constructed, comply with the relevant provisions of the Building Code." *(emphasis in original)*

24. In his submission to Engineering New Zealand dated 29 April 2017, Mr O'Connor told Engineering New Zealand that prior to completing the "documentation" and issuing the PS1 "I would have reviewed the design done by [Mr B] to the extent warranted by his experience and the level of supervision KOA provided". Mr O'Connor said his checks would have included calculations and design, but did not provide any additional or specific details about what his review included in this case. Mr O'Connor told the Investigating Committee that the engineers involved were very experienced and he relied on their review. Mr O'Connor said there is no way of knowing that the senior engineer had completed their check or the extent of that check and he would take the engineer's word that the check had been completed.[8]

25. On 22 June 2011, a building consent application was submitted to Masterton District Council.

26. On 30 June 2011, Masterton District Council wrote to Spencer Holmes Limited requesting a 'screening review'. The request stated: "Please screen the engineers design for construction and compliance with B1 of the building code *(sic).*" Copies of the plans (both the engineering and architectural), and the engineers' calculations and specifications were provided. The PS1 signed by Kevin O'Connor was included in the information provided.

27. On 4 July 2011, Spencer Holmes Ltd wrote to Masterton District Council stating:

[8] Mr O'Connor met with the Investigating Committee in a face to face interview on 26 March 2018.

"We consider the documents provide reasonable grounds for Masterton District Council to accept a producer statement PS1 as a basis to issue Building Consent subject to;

The designer or other approved engineer;

- Verifying that the ultimate ground bearing capacity and foundation conditions are in accordance with design assumptions of 300 kPa ultimate bearing capacity; and
- Construction monitoring of the structural aspects to ACENZ/IPENZ level CM3 and provision of a Producer Statement PS4 — Construction Review for the construction."

28. On 19 July 2011, Masterton District Council wrote to KOA Ltd advising that the Council will issue a building consent "with a condition stating that the issue of code compliance certificate at the end of the project will be reliant on the issue of a producer statement PS4 construction review from you".

29. On 21 July 2011, Mr Z responded on behalf of KOA Ltd confirming that KOA Ltd had been engaged to carry out "construction monitoring of all structural aspects to a CM3 level. Our inspections will include all foundation excavations, pre pours, building steelwork prior to cladding, and mezzanine floor." Mr Z confirmed that KOA Ltd would issue a PS4 on completion of the work.

30. On 26 July 2011, Building Consent was issued.

CONSTRUCTION

31. Building construction commenced mid-2011.

32. On 3 November 2011, Kevin O'Connor signed a PS4 that stated that KOA Ltd was engaged to "provide construction observation services to ACENZ CM3 level in respect of clause(s) B1 STRUCTURE of the Building Regulations for the building work described by the drawings and specifications prepared by KOA titled New Retail Outlet and numbered 111012 S1 Rev. A, S2 Rev. A, S5 Rev. A, S10 — S18 Rev. A, S20 — S24 Rev. A & SD1 — SD2 Rev. been issued."

33. It stated that "[w]e have sighted the Building Consent No. 110268 and the attached conditions of Building Consent" and that the specified building work has been completed "to the extent required by the Building Consent …"

34. According to the Council records, on 4 November 2013 a Code Compliance Certificate was issued. It is unclear from the Council records why there was a delay between construction of the building being completed and the Code Compliance Certificate being issued.

SUBSEQUENT REVIEWS OF BUILDING

35. On 4 August 2014, MTLT wrote to Proarch Architects Ltd, noting that it had become aware of engineering design issues relating to another project that KOA Ltd had provided engineering design for. MTLT requested advice as to whether similar issues may also apply to any of its buildings that KOA Ltd had been involved in. This letter was forwarded to KOA Ltd. On 23 September 2014, Mr O'Connor responded directly to MTLT stating that he had carried out a desktop review of all the relevant

buildings.[9] He stated that following his review, in his opinion, "the buildings … [have] been properly designed and detailed".

36. On 3 November 2014, MTLT responded that a review by Mr O'Connor or any current KOA Ltd employee did not provide the level of necessary comfort from an independent perspective. MTLT requested advice on how this could be achieved and whether it should consider requesting a peer review.

37. On 21 November 2014, Mr O'Connor responded stating that "I fully understand that KOA could be seen to have a financial interest which could be seen as impairing the personal judgement of the reviewers" and recommended that MTLT seek a detailed seismic assessment.

38. It does not appear that any further action was taken by MTLT at that time.

GA HUGHES & ASSOCIATES LTD

39. As noted above, in June 2016, GA Hughes & Associates Ltd undertook a high level structural review of the building. The information considered as part of the GA Hughes & Associates Ltd review included the Masterton District Council property file, including the engineering calculations and drawings. A site visit and visual inspection of the building was also carried out. GA Hughes & Associates Ltd also undertook an Initial Evaluation Procedure (IEP) for the building, which is a quick assessment method based on factors such as the buildings age, the type of materials used, and the construction type, used to identify if a building is potentially earthquake prone.

40. In summary, the review identified concerns with the structural integrity of the building. The report summary stated:

> "We have found little documented justification in the design calculations for the selected ductility factor of 3.
>
> Bracing details and connections are not compatible with the requirements set out in the standards. There is no consideration of the Cs factor in bracing design.
>
> The lateral load resisting system is vulnerable.
>
> There are no clear connections between bracing elements and precast bracing walls on one side.
>
> There are concerns about the capacity of the pile assumed to provide fixity at the portal legs.
>
> There is doubt about the capacity of the transom, fixings and supports under actions as the parts provisions of the loading standards."

41. GA Hughes & Associates Ltd assessed the building, using the IEP assessment, as being 35% of the New Building Standard (NBS).

[9] This included 96-120 Queen Street.

HOLMES CONSULTING

42. In September 2016, Holmes Consulting undertook a Detailed Seismic Assessment of the building, "with review of the structural capacity of vulnerable details in the […] buildings in question as identified by previous reporting by G.A. Hughes".

43. The report noted that the "principal elements of our scope are to provide an independent review of load paths and configuration, as well as key detailing that may affect the seismic capacity of the buildings".

44. The review estimated the building's strength to "35-45% of the strength of an equivalent new building designed to AS/NZS 1170.5:2004". It stated:

> "The governing elements are the embed connections between the precast panels and the upper transom beam, with the embedded anchors having insufficient pull-out/concrete cone failure capacity to sustain the out-of-plane wall loading. The transom beams, a PFC steel section, yield at a demand lower than 35%, however we have allowed that they will have some ability to sustain inelastic deformations beyond yield even if this results in some lateral flexural buckling and torsion. This means that the embed connections, which have a brittle failure behaviour, are likely to be the limiting component which will have significant impact on the buildings ability to sustain seismic demands, and would pose a life-safety hazard.
>
> "The precast concrete panels along the building's east elevation are likely to be unstable in a moderate to large earthquake. This is due to 300 PFC transom beams having insufficient flexural capacity to carry out-of-plane loads resulting from connections to the precast panels. Additionally, connections between the transom beams and precast panels occur on only one side of the beams, which will result in torsion being applied to the beams. …
>
> "In the PFC transoms buckle the out-of-plane displacement of the precast panels will likely increase which will amplify the loading on the threaded inserts connecting the PFC to the panel, and also the connections of the panel bases to the slab-on-grade edge thickening. The failure of the inserts is a brittle mechanism, therefore we consider them more representative of the building capacity, with their pull-out being between 35-45% of current code demand.
>
> "Also in the eastern direction, it appears that the typical 360UB45 columns have been designed to cantilever up from foundation level in order to support lateral loads. The base plate connection shown on the structural drawings has been assessed at 40% of code for ductility of 3.0. …
>
> "Finally, were the column base plate capable of transmitting the design loads into the foundation, it is likely the pile foundation, which measures 2.0 metres long and 750 millimetres in diameter, would fail surrounding soils through bearing, resulting in the foundation 'rocking' around in its pocket within the ground. This rocking would increase drifts within the frame, which already sit at approximately 2% at ULS loads."

EXPERT ADVICE

45. Engineering New Zealand engaged two independent expert structural engineers — Stuart George and Barry Brown — to provide advice on this matter. They were asked to advise whether, in their opinion, the engineering designs were produced in accordance with accepted standards at the time and

consistent with what a reasonable engineer, in the same circumstances, would likely have done. They were not asked to undertake a seismic assessment of the building and neither have undertaken an evaluation of the seismic performance rating. This is a key difference in the focus of the advice provided by Mr George and Mr Brown compared to the assessments undertaken by GA Hughes & Associates Ltd and Holmes Consulting – the Hughes and Holmes reports are instead concerned with the building's current performance.

46. The information Mr Brown and Mr George considered in forming their opinions included the Masterton District Council property file, Mr O'Connor's submissions, as well as the GA Hughes and Associates Ltd and Holmes Consulting reports.

47. In summary, both experts identified concerns with the design of the building. Mr Brown, considered that overall the structural concept for this building was reasonably well developed but identified concerns with the lateral resisting system. Mr Brown stated:[10]

> "The most serious defect in the primary system is the capacity of the cast-in RB base connection of the precast concrete wall panels to the foundation slab, which (although backed up by the eaves level transom) appears to me to be vulnerable due to under capacity."

48. Mr Brown advised that according to his calculations these connections will likely be subject to both tension and shear demand, and "as such are unlikely to exhibit the required capacity to satisfy the relevant design standards, eg NZS 3101:2006 section 17".

49. Mr George noted that there are one-legged portal frames supported by slender precast concrete walls which is an unusual load resisting system and not a robust load path.[11] Mr George considered that the slab thickening was unlikely to have sufficient weight to be effective.

50. In assessing whether the overall design met acceptable standards Mr George stated:[12]

> "The definition of meeting acceptable standards is difficult to define. My best analogy would be marking a project at Engineering School, it is inevitable that [there] will be a wide variation in marks, and unlikely that anyone would score 100%. Even with the benefit of years of experience and an engineering registration process engineering knowledge and judgement varies."

51. Mr George gave the building at 96-120 Queen Street an overall grade "C".

52. See **Appendix A** and **B** for a full copy of their reports.

RESPONSE FROM MR O'CONNOR

RESPONSE TO COMPLAINT

53. In a statement to Engineering New Zealand dated 29 April 2017, Mr O'Connor stated that, in his opinion, "the building complied with accepted design standards". Mr O'Connor stated that this has been confirmed through a number of reviews carried out by KOA Ltd, including a review "using an

[10]See Mr Brown's report dated 31 October 2017.

[11]See Mr George's report dated 11 August 2017.

[12] See Mr George's report dated 11 August 2017.

equivalent static analysis"[13], completed by a senior KOA Ltd engineer, which "showed compliance". He said that KOA Ltd had also rechecked a number of the elements of the design to ensure they are adequate. Mr O'Connor also advised that a review, carried out by a senior KOA Ltd engineer, using a "3-Dimensional Non-Linear Push-over Analysis (NLPA) confirmed the design of 96-120 Queen Street complies to 100% NBS using ductility factor of 1.25." Mr O'Connor has provided the results of the NLPA, and some calculations for the transom capacity and foundation moment capacity.

Mr O'Connor's comments on the GA Hughes & Associates Ltd report

54. In relation to the GA Hughes & Associates Ltd report, Mr O'Connor noted that this is an IEP which, as stated in the report, is based only on a document review and visual inspection of the building. Mr O'Connor stated that "[t]his type of preliminary report cannot be relied on to be accurate". Further to this Mr O'Connor stated:

> "It [the report] comments on the adequacy of numerous items, without analysis, to demonstrate there are valid concerns in relation to; transoms, portals and roof bracing adequacy."

55. Mr O'Connor stated:

> "…for many years roof bracing has been designed without concerns for large eccentricities and this has never been shown to be a problem in causing failures in single storied buildings."

56. Mr O'Connor also noted that the report references the SESOC "INTERIM DESIGN GUIDANCE VERSION 4", which was not issued at the time of the design of this building.

57. In relation to the comment that "the portals, transoms and panel fixings need detailed assessment in respect of the parts provisions of the design standards" Mr O'Connor submitted that "it was common amongst engineers to not design precast panels supported on the ground to the parts provisions". Mr O'Connor referred to a MBIE determination,14 and NZS 1170 and NZS 4203 and submitted that from these documents "it can be seen that precast panels, which are load bearing and provide structural resistance would not have been deemed a part and that the common practice of not designing these elements as parts on single storey buildings to NZS 4203 has continued with the introduction of NZS 1170". Mr O'Connor also submitted that the NLPA carried out by KOA Ltd "has confirmed that the building is compliant and collapse will not occur".

58. In relation to the comment about no fly-bracing being present, Mr O'Connor submitted that this is not necessary and that the NLPA carried out be KOA Ltd confirms this.

59. In relation to the comment that there are "significant eccentricities", Mr O'Connor stated:

> "Presumably, these are comments about Cs factors in the roof bracing. The building is 8m wide in the longitudinal direction and 40m long thus it is debatable whether any bracing is required. … we have an 8m wide building 40m long supporting lightweight roof and walls and thus in the long axis of the building, the load will pass through the roofing, purlins and portals bending about their weak axis and with bracing albeit eccentric."

60. Mr O'Connor submitted that this demonstrates a "rigid application of the Code" and goes on to state:

[13] Equivalent static analysis is a method used to analyse the seismic performance of a building.

[14] 2013/057.

> "…using this to say the building is irregular and eccentric and then to apply overly conservative factors in the IEP has led to grossly understating the buildings %NBS as 35% when it should be 100%."

61. In relation to the recommendation that the "capacity of the as constructed building in the transverse direction need to be considered in more detail" Mr O'Connor stated that the KOA Ltd NLPA demonstrates that the building is compliant.

62. In relation to the recommendation that the "[c]apacity of fixing at piles and base portals need checking", Mr O'Connor stated that the foundation material is gravel which have "high STP or CPT values". Mr O'Connor stated "the failure of these piles by rotating on the ground seems implausible" and suggests that the comments are based on the assumption that the ground is soft.

63. In relation to the IEP, Mr O'Connor commented that he cannot understand why the building shape is being viewed as irregular.

Mr O'Connor's comments on the Holmes Consulting report

64. In his statement to Engineering New Zealand dated 29 April 2017, Mr O'Connor stated that he is "not clear that this [the Holmes Consulting report] is a Detailed Seismic Assessment". Mr O'Connor then responded to each of the recommendations made with respect to new structural elements being required, stating that none of these recommendations are necessary as supported by the NLPA that KOA Ltd has carried out.

65. In relation to the comments on the sufficiency of the foundations, Mr O'Connor again referred to the gravel soils, stating the suggestion that the piles will rock in the ground to be "extremely unlikely" and "implausible" and based on the incorrect assumption that the grounds are soft.

66. In relation to the baseplates for the portals being assessed at 40% of Code for a ductility of 3.0 and not meeting the requirements of NZ 3604 for a category 2 member, Mr O'Connor stated that a ductility of 1.25 was used in their NLPA which shows it to meet the demand.

General comment from Mr O'Connor

67. In general, in relation to both the GA Hughes & Associates Ltd and Holmes Consulting reports Mr O'Connor stated:

> "The reports question the adequacy of a) eccentric transom connections and b) panel connections to foundations without presenting any justification or evidence for these views. Our review and observations shows these details are adequate and further are common details found in many buildings designed by many other engineers."

68. Mr O'Connor provided examples of details for transom precast panel fixings and precast panel/footing details that KOA Ltd used in this and the other buildings. Mr O'Connor submitted that these have also been used by Holmes Consulting in more recent buildings. Mr O'Connor submitted that a level of judgement is needed when carrying out an assessment and that "[t]hose rigidly applying the codes and using an ESA (sic) would most likely come up with the lowest results and those using judgement somewhat better and then those using nonlinear analysis better again".

69. Furthermore, in an interview with the Investigating Committee on 26 March 2018, Mr O'Connor reiterated his view that, while some fly bracing could be added to provide additional support to the building, it was safe and that this had been confirmed in the NLPA assessment. However, Mr O'Connor told the Investigating Committee that in hindsight, he should not have signed the PS1 at the time.

RESPONSE TO PROVISIONAL DECISION

70. In response to the provisional decision Mr O'Connor noted that in its decision the Investigating Committee is considering his actions in relation to the signing of the PS1, but has not identified what accepted standards in relation to the use of PS1s are. Mr O'Connor referred to Engineering New Zealand's Practice Note 1 Guidelines on Producer Statements,[15] noting it states that "limited standardised practice has developed" in relation to the use of PS1s and that a producer statement is "not a product warranty or guarantee of compliance".

71. Further to this, Mr O'Connor reiterated his view that, based on the information he had available when signing the PS1, he had reasonable grounds to believe that the designs complied with the Building Code. Mr O'Connor stated:

> "In signing the PS1 I confirmed that I had a reasonable basis for thinking that, in my opinion, the designs complied with the Building Code. I did have a reasonable basis for thinking the designs complied with the Building Code. … [i]t was my practice to check aspects of the calculations in the designs, and to place weight on the fact the designs had been prepared and/or reviewed by an engineer with suitable experience and the person doing this work had 20 years experience as an engineer and the building is a relatively low risk structure smaller than most houses."

72. Mr O'Connor noted that there was no requirement for him to "to exhaustively review the designs and calculations for the purpose of preparing the PS1".

73. In addition, Mr O'Connor expressed concern regarding the conclusion in Mr Brown's advice report of 30 April 2018, in which Mr Brown stated: "if the output PS1 for design … signed off by the CPEng/Principal is defective in terms of its statement … the author must accept responsibility arising from that." In relation to this statement Mr O'Connor stated:

> "This is not the inevitable consequence of a defective design, if the author of the producer statement-took reasonable steps before signing it. A producer statement states that the author believes on reasonable grounds etc… Thus the issue is whether reasonable steps were taken, but in hindsight if those steps are found to reveal an error may have occurred that does not make the process unreasonable."

74. Mr O'Connor stated that the environment and standards have changed significantly since the Canterbury earthquakes. Mr O'Connor stated:

> "The environment we work in now as engineers and the standards expected have significantly changed because of the scrutiny applied to the profession since the Christchurch earthquakes. I believe if such scrutiny was applied to engineering work prior to this time, many practices and design details when judged by today's standards, would be discovered and deemed deficient."

[15] See: www.engineering nz.org/resources/practice-notes-and-guidelines

75. Mr O'Connor reiterated his submission that many of the details that have been identified as being deficient in these cases were commonplace at the time the buildings were designed.

DISCUSSION

INVESTIGATING COMMITTEE'S ROLE AND THE LEGAL CONTEXT

76. Professional disciplinary processes primarily exist to protect the public, uphold professional standards, and maintain public confidence in the profession and its regulation. They do this by ensuring that members of the profession adhere to certain universal (or accepted) professional standards.[16]

77. Our role in this professional disciplinary process is to determine whether there are grounds to dismiss the matter in rule 57(a) of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002. If none of these grounds to dismiss the complaint apply, then the complaint must be referred to a Disciplinary Committee in accordance with rule 60(a) of the Rules.

78. In order to determine whether there are grounds to dismiss the matter, we have investigated whether Mr O'Connor provided engineering services relating to the engineering design work on 96-120 Queen Street in accordance with accepted professional standards at the time the building was designed. We are not concerned with whether the building strictly complied with the building code or the building's current seismic rating – our assessment is whether Mr O'Connor acted reasonably at all stages of his involvement in the design of this building, including when he reviewed the design and signed the PS1.

79. The legal test we need to assess Mr O'Connor's actions against is whether he acted reasonably and in accordance with what a reasonable body of his peers would have done in the same situation (i.e. did he act in accordance with accepted standards).

80. If the evidence is that Mr O'Connor acted in accordance with accepted standards then we can dismiss the matter.[17] If the evidence is that Mr O'Connor did not act in accordance with accepted standards then we need to assess how significant his departure from accepted standards was – if it was minor, we may dismiss the matter as insufficiently grave to warrant further investigation;[18] if it is more than minor, and no other ground for dismissal applies,[19] we are required to refer the matter to a disciplinary committee.

81. The issue we have specifically considered in this case is whether in signing the PS1 Mr O'Connor was acting in accordance with accepted professional standards at the time the building was designed.

[16] *Dentice v Valuers Registration Board* [1992] 1 NZLR 720 (HC).

[17] Rule 57(a) of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002, on the basis that there is no applicable ground of discipline under section 21(1)(a) to (d) of the Act.

[18] Rule 57(ba) of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002.

[19] There are other grounds in the legislation for dismissing a complaint, including: where the subject matter of the complaint is trivial; the complaint is frivolous or vexatious or is not made in good faith; where the person alleged to be aggrieved does not wish action to be taken or continued; where the complainant doesn't have a sufficient personal interest in the subject matter of the complaint; or an investigation is no longer practicable or desirable given the time elapsed since the matter giving rise to the complaint (Rule 57 of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002. In our view, given the nature of the Inquiry, we cannot apply any of these grounds to reasonably dismiss this matter.

REVIEW AND SIGN OFF OF THE DESIGN BY WAY OF PS1

Adequacy of the design

82. Mr O'Connor submitted that the building complied with accepted design standards and is safe, which he says has been confirmed through a number of reviews and checks conducted by senior KOA Ltd engineers using different methods, most recently a NLPA.

83. In contrast, both Mr George and Mr Brown identified concerns with the engineering design of this building, primarily relating to the lateral resisting system and wall panel/foundation slab joint connections.

84. Mr Brown advised that the most serious defect is the base connection of the precast concrete wall panels to the foundation slab "which … appears to me to be vulnerable due to under capacity" and "unlikely to exhibit the required capacity to satisfy the relevant design standards, eg NZS 3101:2006 section 17". Mr Brown considered that this defect was moderately serious.[20]

85. Mr George advised that "[u]sing slender PC panels as portal legs is not a robust load path".[21]

86. In response to the provisional opinion, Mr O'Connor submitted that many of the details that have been identified as being deficient were commonplace at the time of the building's design and are the same as those used by Holmes Consulting in recent examples. Mr O'Connor stated:

> "The environment we work in now as engineers and the standards expected have significantly changed because of the scrutiny applied to the profession since the Christchurch earthquakes. I believe if such scrutiny was applied to engineering work prior to this time, many practices and design details when judged by today's standards, would be discovered and deemed deficient."

87. As noted above, our experts were asked to advise whether, in their opinion, the engineering designs were produced in accordance with accepted standards at the time and consistent with what a reasonable engineer, in the same circumstances, would likely have done. While the GA Hughes and Holmes Consulting reports focus on the current performance of the buildings and the issues identified relate to current standards, Mr George and Mr Brown's advice relate to the accepted standards at the time the buildings were designed – 2011 in the case of 96-120 Queen Street.

88. We note that both the experts considered all the information provided to Engineering New Zealand by Mr O'Connor, including his submission of 29 April 2017, as well as a summary of the Investigating Committee's interview with Mr O'Connor on 26 March 2018.

89. Mr O'Connor has provided us with no new information. Accordingly, guided by our experts, we confirm our concern that there are deficiencies with the engineering design of this building that indicate it was not of an adequate standard, taking into account the year that it was designed.

Should Mr O'Connor have signed the PS1?

90. Chartered professional engineers use PS1s to confirm their professional opinion that aspects of a building's design comply with the Building Code. Their intent is to signal to a building consent authority

[20] See report dated 31 October 2017.

[21] See report dated 11 August 2017.

(BCA) that certain design work has been done (or overseen/supervised) by a practitioner who is competent to perform the defined work.[22]

91. Although they have no legal status under the Building Act 2004, PS1s are intended to provide BCAs with information to establish that there are reasonable grounds for the issue of a building consent. When used properly, they give BCAs confidence that certain building work will be constructed to meet the Building Code.

92. Chartered professional engineers should be aware that BCAs are likely to rely on producer statements to some extent, and therefore should be mindful that responsibilities and potential liabilities may arise from signing them.[23]

93. In signing the PS1 for the building Mr O'Connor was confirming his professional opinion that aspects of the building's design complied with the Building Code. Accordingly, by signing the PS1 Mr O'Connor assumed responsibility for the building's design.

94. Therefore, the question for us to consider is whether it was reasonable for Mr O'Connor to sign off the building's design by way of PS1, or whether he should have identified and responded to the issues identified by our experts before providing his sign off. In other words, the question for us is whether Mr O'Connor carried out his part in the review and sign off process in a careful and competent manner and in accordance with accepted professional standards.

95. The starting point for this assessment is to consider Mr O'Connor's role in the review and sign out process.

96. Mr O'Connor told Engineering New Zealand that his role in the design and development of the building was limited to the final review and sign off of the design by way of PS1.

97. Mr O'Connor told Engineering New Zealand that prior to completing the "documentation" and issuing the PS1 "I would have reviewed the design done by [Mr B] to the extent warranted by his experience and the level of supervision KOA provided". Mr O'Connor said his checks would have included review of the calculations and design. He did not provide any additional or specific details about what his review included in this case. He said he was guided by the senior engineer involved carrying out their structural check of the designs. Mr O'Connor said that the senior engineer in this case was very experienced but that he had no way of confirming the check had been completed or to what extent – he was reliant on the senior engineer's word.

98. Mr O'Connor noted in his response to the provisional decision that there is no standard practice in relation to the use of PS1s and he refers to Engineering New Zealand's Practice Note 1 *Guidelines on Producer Statements* (the Practice Note) to support this. He also said that there was no requirement for him to "exhaustively review the designs and calculations for the purposes of preparing the PS1". Mr O'Connor submitted that based on his review and assurance provided by the senior engineer involved he had "a reasonable basis for thinking the designs complied with the Building Code".

[22] Engineering New Zealand Practice Note 1 *Guidelines on Producer Statements* (January 2014), section 3.1.

[23] Ibid.

99. The part of the Practice Note that Mr O'Connor refers to in support of his argument about standard practice states: "By the same token there is no provision for BCAs to require a producer statement, as of right. As a result of their non-mandatory status, limited standardised practice has developed. There is currently such a wide variation in the way BCAs accept and/or rely on producer statements that there is a resultant degree of confusion, frustration and inefficiency amongst practitioners and BCAs." We interpret the Practice Note to be saying that the lack of standardised practice related to the extent by which producer statements are used by BCAs; not the extent of engineering practice with regard to the level of review a chartered professional engineer should carry out before signing off a PS1 – which is the issue in this case. We consider that, in 2011, there would be a clear expectation from Mr O'Connor's peers that a chartered professional engineer signing off a design by way of PS1 would take sufficient steps in their review of a design to satisfy themselves that they had reasonable grounds for signing the PS1.

100. We accept there was no requirement to undertake an "exhaustive" review himself. However, as noted above, in signing the PS1 Mr O'Connor was confirming his professional opinion that aspects of the building's design complied with the Building Code. Regardless of Mr O'Connor's trust in the experience and competence of the designer or the senior engineer who carried out the checks, as a CPEng engineer and the person signing out the designs, Mr O'Connor had a responsibility to carry out an adequate review himself, and not just rely on the reviews that may or may not have been carried out at other steps of the system. As noted by Mr Brown:[24]

> "if the output PS1 for design … signed off by the CPEng/Principal is defective in terms of its statements (and irrespective of the reliance placed on verbal advice given by a Senior Engineer), the author must accept the responsibility arising from that. Put another way, the process that the CPEng/Principal used in reaching his/her judgement regarding the sufficiency of building design documentation is irrelevant if the statement is defective."

101. We note that in response to the provisional decision Mr O'Connor argued that if the author of a PS1 took reasonable steps to check the design met the relevant standards, if an error is later identified, "that does not make the process unreasonable".

102. We agree with Mr O'Connor that the issue is whether he took reasonable steps to check that designs.

103. Mr George advised that the issues in the design should have been identified during the checking stages of the design development.[25] There is no evidence that they were.

104. Overall, when weighing up the advice from the experts, it is our opinion that Mr O'Connor did not act in accordance with accepted standards in signing off this design by way of a PS1. As noted by Mr Brown, the issues identified may affect the building's capacity to withstand a severe seismic event. We consider that, as the CPEng engineer signing off the design, Mr O'Connor should have identified the issues as noted by our experts and, in the very least, taken additional steps in light of those issues to satisfy himself that the building met relevant standards before signing the PS1. That may have included, for example querying this with the design engineer or senior reviewing engineer. There is no evidence that he did identify and respond to these issues.

[24] See report dated 30 April 2018.

[25] See report dated 19 April 2018.

105. Furthermore, in our view, a reasonable engineer would document any process of checking and clarifying aspects of a design at the sign-off stage, and included documentation to support their assertions.

106. In our opinion, a reasonable body of Mr O'Connor's peers would likely consider that his actions in signing the PS1 (and without identifying and responding to the design issues which have been identified by our experts), was not consistent with accepted practice at the time. In these respects, we consider there is an applicable ground of discipline in this case, and we do not consider that any of the grounds to dismiss this matter apply. Whether Mr O'Connor's actions are a breach of his professional and ethical obligations and reach the threshold for professional discipline is a matter for a disciplinary committee.

107. We note that Mr O'Connor told the Investigating Committee that in hindsight, he should not have signed the PS1 at the time and the building could have benefitted from some additional fly bracing to provide additional support.

CONCLUSION

108. Overall, we consider that it was inappropriate for Mr O'Connor to sign the PS1 in the circumstances described above and as set out in the information considered by us above. In our opinion, Mr O'Connor should have identified the issues noted by our experts when reviewing the design for sign-off and taken additional steps to reassure himself the designs met the relevant standards before signing the PS1. There is no evidence he did this. We consider this was more than a minor departure from accepted standards.

109. Accordingly, we do not consider that there are grounds to reasonably dismiss the matter on the information collected to date and have decided to refer it to a disciplinary committee in accordance with rule 60(a) of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002.

Andrew McMenamin
Chair of Investigating Committee

Stewart Hobbs
Dr Sulo Shanmuganathan
Committee Members

INVESTIGATING COMMITTEE DECISION

OWN MOTION INQUIRY ABOUT KEVIN O'CONNOR — 408 QUEEN STREET

For release

In accordance with:

Engineering New Zealand Rules
Engineering New Zealand Disciplinary Regulations

Issued by
Andrew McMenamin CPEng CMEngNZ
Chair of Investigating Committee

Stewart Hobbs CPEng FEngNZ IntPE(NZ)
Dr Sulo Shanmuganathan CPEng FEngNZ
Committee Members

25 March 2019

engineering
new zealand
Institute of Engineering Professionals

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BACKGROUND

1. In 2015, Engineering New Zealand[1] was made aware of concerns surrounding six buildings in Masterton owned by Masterton Trust Lands Trust (MTLT). The concerns were raised by a Chartered Professional Engineer (CPEng).

2. The building, located at 408 Queen Street was one of the buildings identified.

3. The concerns were about the structural integrity of the buildings. Because Engineering New Zealand has no jurisdiction over physical assets, Engineering New Zealand brought the concerns to the attention of the Ministry of Business, Innovation and Employment (MBIE), as the relevant regulatory authority.

4. MBIE subsequently commissioned GA Hughes & Associates to carry out a structural review of the buildings,[2] which identified concerns with the buildings and recommended a Detailed Seismic Assessment be carried out. Following the receipt of this report, MTLT and MBIE commissioned a Detailed Seismic Assessment to be carried out by Holmes Consulting, which identified concerns about the structural integrity of the six buildings. MTLT subsequently commenced remedial works on some of the buildings.

5. In light of the findings of these two reports Engineering New Zealand decided that it needed to act on this information to determine if there is an issue with the engineering design of these buildings or not and, if so, what that means. It was decided that the best way to do this was by way of an own-motion inquiry pursuant to Rule 55 (1) of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002 (the Rules).[3]

SCOPE OF INQUIRY

6. The scope of the Inquiry was to conduct an initial investigation into:

   > "The circumstances relating to the design, design review and construction monitoring of six buildings in Masterton:
   >
   > - to assess whether the Chartered Professional Engineers involved have provided engineering services in accordance with accepted standards; and
   > - to learn and advise on any engineering performance and practice improvements, if necessary."

[1] On 1 October 2017 IPENZ changed its name to Engineering New Zealand. Accordingly, I will refer to Engineering New Zealand throughout this report.

[2] One of the buildings that Engineering New Zealand was initially notified about was different from the six ultimately chosen by MBIE to assess further.

[3] It has subsequently been identified that the design of this building pre-dates the Rules, and that Mr O'Connor was not a Chartered Professional Engineer at that time. However, he was a Professional Member of IPENZ and, accordingly, this matter was re-notified in accordance with clause 4(1) and clause 9(a) of the Engineering New Zealand Rules and Disciplinary Regulations.

7. Kevin O'Connor & Associates (KOA Ltd) was the engineering firm involved in the engineering design of all six of the buildings. Kevin O'Connor CPEng, CMEngNZ[4], IntPE(NZ)[5], was identified as the specific engineer involved in the sign off of the engineering design of the building at 408 Queen Street.

8. The building at 408 Queen Street was designed and built in 2003. Mr O'Connor assumed overall responsibility for the design when he signed the Producer Statement – PS1 - Design for this building on 24 February 2003. At that time, Mr O'Connor was not a Chartered Professional Engineer. Mr O'Connor first became registered as a Chartered Professional Engineer on 6 October 2004. For this reason, Mr O'Connor's involvement is being considered in relation to his capacity as a member of Engineering New Zealand and his obligations under the Engineering New Zealand Rules, as they applied at the time.

9. The issue being considered with respect to Mr O'Connor is:

- Whether Kevin O'Connor provided engineering services relating to the engineering design work on the building at 408 Queen Street in accordance with accepted standards.

INVESTIGATION

10. Following an initial investigation this matter was referred to an Investigating Committee for formal investigation. The Investigating Committee is:

    Andrew McMenamin CPEng CMEngNZ (Chair)

    Dr Sulo Shanmuganathan CPEng FEngNZ

    Stewart Hobbs CPEng FEngNZ IntPE (NZ)

11. Information collected by Engineering New Zealand for the purposes of this investigation included information from:

Kevin O'Connor	Respondent/engineer
Kevin O'Connor & Associates Ltd	Engineering company
Masterton District Council	Local authority
Masterton Trust Lands Trust	Building owner
GA Hughes & Associates	Consulting structural engineering company
Holmes Consulting	Consulting structural engineering company

12. Independent expert advice was obtained from Barry Brown CPEng, FEngNZ[6], IntPE(NZ) (**Appendix A**) and Stuart George CPEng, CMEngNZ, IntPE(NZ) (**Appendix B**).

[4] Chartered Member of Engineering New Zealand.

[5] A member of the New Zealand section of the International Professional Engineers.

[6] Engineering New Zealand Fellow.

INFORMATION GATHERED

13. Masterton Trust Lands Trust (MTLT) engaged Murphy Properties Ltd to develop a new commercial/retail premises, located at 408 Queen Street. Proarch Architects Ltd were engaged to undertake the architectural design for the development.

14. According to Kevin O'Connor, in February 2003 Murphy Properties Ltd engaged Kevin O'Connor & Associates Ltd (KOA Ltd) to provide the engineering design and draughting for the development. Mr O'Connor told Engineering New Zealand that KOA Ltd had a standing "annual agreement" with Murphy Properties Ltd and so there is no letter of engagement or short form contract for this project.

15. Mr O'Connor advised that the design brief was provided to KOA Ltd by Proarch Architects Ltd. MTLT told Engineering New Zealand that Proarch Architects Ltd acted as the project manager and all communications with KOA Ltd and the Masterton District Council went through Proarch Architects Ltd.

16. In relation to the extent of the briefing they would have received from Proarch Architects Ltd, Mr O'Connor told the Investigating Committee that KOA Ltd would have received an initial briefing letter from Proarch Architects Ltd with a set of sketch plans but would not have had any further information than this.

BUILDING DESIGN

17. The building is a single level commercial/industrial building, constructed with a structural steel portal frame and precast concrete panels, with a metal roof. The foundation is a 150mm concrete slab on grade, incorporating edge thickening.

ENGINEERING DESIGN

18. Mr O'Connor told Engineering New Zealand that after being engaged to complete the designs he "set up the job, conducted the detailed design work and organised the draughting". Mr O'Connor told the Investigating Committee that the process at KOA Ltd at that time was for the draughting to be carried out by a draughtsperson, and a senior engineer would carry out a structural check, which would then be reviewed by Mr O'Connor. Mr O'Connor said that he was not directly involved in the design development of this building but would have carried out the final review prior to sign out of the designs.

19. Mr O'Connor said that at the time the building was designed they would "typically" have been using the structural analysis software, Microstran.

20. On 24 February 2003, Mr O'Connor signed a Producer Statement — PS1 — design (PS1) which states:

> "I BELIEVE ON REASONABLE GROUNDS that subject to:
>
> i. The site verification of the following design assumptions N/A; and
>
> ii. All proprietary products meeting the performance specification requirements, the drawings, specifications, and other documents according to which the building proposed to be constructed comply with the relevant provisions of the building code."

21. The PS1 refers to sheets S1-S8a, but does not record the revision. The plans held on the Masterton District Council file are undated, and marked as S1-S8a, revision “P1”. The calculations are dated 24 February 2003.

22. In a letter dated 24 February 2003 from Proarch Architects to Masterton District Council, the documents that Proarch Architects were “submitting for Building Consent” are listed, and these included KOA Ltd drawings “S1-S8a”. The PS1 was not listed in this set of documents. Mr O’Connor has since told the Investigating Committee that on review of the file held by KOA Ltd it appears the PS1 was sent to Proarch Architects Ltd on 25 February 2003 and was not accompanied by any plans.

23. By signing the PS1 Mr O’Connor signified that he had reasonable grounds to believe that the building would comply with the relevant provisions of the Building Code (as it applied at the time).

24. In relation to quality assurance processes Mr O’Connor stated: “the plans would have been subject to a normal dimensional check and a structural check by myself and gone through the normal council review process before being issued.”

FURTHER REVISIONS CARRIED OUT

25. Mr O’Connor told the Investigating Committee that from the KOA Ltd file it is apparent that the plans were revised following the Producer Statement being issued on 24 February 2003. Mr O’Connor said it was not unusual for this to occur at the time this building was being designed.

26. Mr O’Connor told the Investigating Committee that it appears from correspondence held on the KOA Ltd file that the architect corresponded with a junior KOA Ltd staff member and major changes were made to the design after 28 February 2003. In particular, on 3 March 2003 the building length was reduced by approximately two metres (from 26837 mm to 25093 mm), the end bay of the building reduced to 7215mm, and the panels were adjusted accordingly. Mr O’Connor said that the building layout was also changed so that there were three equal bays but the panels were not changed, and building services were added. A copy of these changes are not held on the Masterton District Council file.

27. Mr O’Connor provided Engineering New Zealand with a copy of the ‘check set’ of the revised plans. The dates are blacked out on the plans. However, Mr O’Connor said he believes that these changes were made on or around 3 March 2003. Mr O’Connor said he does not know why the dates have been blacked out. These plans include the change to the building length, from 26837mm to 25093mm.

28. Mr O’Connor also provided the Investigating Committee with a copy of a fax from Proarch Architects to himself which includes a hand-written response written and signed by him, dated 1 March 2003. The fax from Proarch Architects states:

> “… all the steel for the panels is going to arrive Monday. With the panel changes can you keep changes to a minimum in terms of steel where building is cut back?
>
> “We think this will not be a huge issue because effectively we are altering panels each side where the building is cut back in overall length …”

29. A handwritten response which is signed “Kevin” at the bottom of the fax states:

> “Unfortunately I emailed [KOA Ltd staff member] the drawings 1st thing Saturday and he’s changed them to have 3 equal Bays. I’ve got some minor edits to do so I’ll get them to you.”

30. Mr O’Connor told Engineering New Zealand that additional changes were then made to the design of the building. Mr O’Connor provided Engineering New Zealand with a copy of these revised plans (S1 rev C, S2 rev B, S3 rev C, S4-S5 rev B, S6 rev C, S7 rev B, S8 rev C, S9 rev P1), which are dated as having been changed on 3 March and 6 March 2003.

31. Mr O’Connor said that on review of these plans there is an obvious error relating to the transom, which is not in the earlier revisions. Mr O’Connor originally told the Investigating Committee that the review and sign out of the engineering design was his responsibility, but it does not appear (and there is no evidence) that he carried out a review of the final set of designs which incorporated these changes. Mr O’Connor stated that, as a result, this error “slipped through the cracks”.

32. Neither of the revised plans are held on the Masterton District Council file and there is no evidence of the building consent ever being amended. Furthermore, no further Producer Statement was issued relating to the revised set of designs.

33. Mr O’Connor said that he is not sure what information was provided to Masterton District Council as KOA Ltd had no involvement with this part of the process.

STRUCTURAL CONCEPT

34. Lateral and earthquake loads are resisted by in-plane action of the precast concrete panels and structural steel bracing. The boundary wall face loads are resisted by the precast concrete panels that have been designed to cantilever and also resisted by the thickened foundation concrete slab.

BUILDING CONSENT

35. On 25 February 2003, a Building Consent application was submitted to Masterton District Council.

36. On 12 March 2003, Building Consent (No: 030125) was issued by Masterton District Council. While the consent document records the application date as 25 February 2003, it is unclear whether the consent was granted in relation to the original set of plans or the revised set of plans dated 3 and 6 March 2003.

CONSTRUCTION

37. Construction was carried out in 2003. It is unclear from the property file whether the building was built in accordance with the original (undated) designs or the revised designs dated 3 and 6 March 2003.

38. The Council carried out inspections during construction in 2003, including foundation, pre-slab, concrete floor, pre-lining, and bracing.

39. On 4 September 2003, a Code Compliance Certificate was issued.

SUBSEQUENT REVIEWS OF DESIGNS

40. On 4 August 2014, MTLT wrote to Proarch Architects Ltd, noting that it had become aware of engineering design issues relating to another project that KOA Ltd had provided engineering design for. MTLT requested advice as to whether similar issues may also apply to any of its buildings that KOA Ltd had been involved in. This letter was forwarded to KOA Ltd. On 23 September 2014, Mr O'Connor responded directly to MTLT stating that he had carried out a desktop review of all the relevant buildings. He stated that following his review, in his opinion, "the buildings … [have] been properly designed and detailed".

41. On 3 November 2014, MTLT responded that a review by Mr O'Connor or any current KOA Ltd employee did not provide the level of necessary comfort from an independent perspective. MTLT requested advice on how this could be achieved and whether it should consider requesting a peer review.

42. On 21 November 2014, Mr O'Connor responded stating that "I fully understand that KOA could be seen to have a financial interest which could be seen as impairing the personal judgement of the reviewers" and recommended that MTLT seek a detailed seismic assessment.

43. It does not appear that any further action was taken by MTLT at that time.

GA HUGHES & ASSOCIATES LTD

44. As noted above, in June 2016, MBIE engaged GA Hughes & Associates Ltd, who carried out a 'High Level Structural Review'. The information considered as part of the Hughes review included the Masterton District Council property file, including the engineering calculations and drawings. A site visit and visual inspection of the building was also carried out. GA Hughes & Associates Ltd also undertook an Initial Evaluation Procedure (IEP) for the building, which is a quick assessment method based on factors such as the buildings age, the type of materials used, and the construction type, used to identify if a building is potentially earthquake prone.

45. the review identified a number of concerns with the building. In summary, the Hughes report stated:

> "The building has been designed with cantilever side wall concrete panels. We have concerns about the use of cast in TCM inserts at the cracked zone for opening face loads. There is doubt that an adequate load path between the lower row of inserts and the stabiliser foundation can be achieved. We have concerns about displacement compatibility between the cantilevered wall panels and the steel portal frames. We also have concerns about the load paths, capacity and connections between the upper gable end wall, the PFC eaves strut/tie and SHS brace on one wall and the concrete panels on the boundary wall."

46. More specifically, the review noted that there are "no fly braces and little restraint for the UB rafters and legs". It considered that there were "significant eccentricities in the longitudinal direction", and there is "no support for side walls near the top and lack of clear load paths and connections". The review also commented that the roof bracing is located near the middle of the building and "may not be adequate".

HOLMES CONSULTING

47. In September 2016, Holmes Consulting was engaged by MBIE to carry out a Detailed Seismic Assessment, which included "a detailed drawing assessment, a brief review of supplied calculations to check assumptions for seismic design inputs such as ductility assumptions, and specific calculations to assess the capacity of elements and connections identified as limiting the seismic capacity of each building".

48. Holmes Consulting estimated the building's strength to be 25-33% NBS. In the report it stated:

> "The governing element is the purlin end bolted connections sustaining combined axial and bending demands due to the precast panel restraining forces and gravity loading. The light-gauge purlins are eccentric to the panel anchor and have a low tear-out capacity which would mean the panels lose their top restraint and would significantly compromise the seismic capacity of the building. The panels do not have sufficient cantilever capacity to resist seismic loads with only their base fixity."

49. It also considered that "the frame will likely suffer local and element buckling as a result of the inelastic response imposed by the displacement demands", but while this was undesirable "it is unlikely to significantly affect the gravity load carrying capacity of the portal frames".

50. On analysis of the structural drawings it was also noted that the precast concrete walls along the north and south elevations "appear to have an insufficient amount of out-of-plane restraint at eave level" and the "estimated capacity is 40-50% of the current code demand". It also noted that "[c]onnections between supporting lateral elements and the roof tension bracing and 125 SHS bracing along the south elevation are eccentric and likely to impose secondary moments on the adjacent elements and their connections". (sic)

EXPERT ADVICE

51. Engineering New Zealand engaged two independent expert structural engineers — Stuart George and Barry Brown — to provide advice on this matter. They were asked to advise whether, in their opinion, the engineering designs were produced in accordance with accepted standards and consistent with what a reasonable engineer, in the same circumstances, would likely have done. They were not asked to undertake a seismic assessment of the building and neither have undertaken an evaluation of the seismic performance rating. This is a key difference in the focus of the advice provided by Mr George and Mr Brown compared to the assessments undertaken by GA Hughes & Associates Ltd and Holmes Consulting – the Hughes and Holmes reports are instead concerned with the building's current performance.

52. The information Mr Brown and Mr George considered in forming their opinions includes the Masterton District Council property files, Mr O'Connor's submissions, as well as the GA Hughes & Associates Ltd and Holmes Consulting reports.

REVIEW OF INITIAL DRAWINGS, AS HELD ON COUNCIL FILE

53. Mr Brown and Mr George initially provided their advice based on a review of the information held on the Masterton District Council property file which included the undated engineering designs - S1-S8a, revision "P1". In summary, both experts noted concerns with the designs. Mr George noted significant concerns with the brace details, lack of eaves tie, lack of fly braces, light vertical wall reinforcement, and panel type C fixings. He considered that these issues should have been identified during the design and review process, and that most of them should have been identified during the high-level review prior to the issue of the PS1. He said that the PS1 should not have been signed in this case.

54. Mr Brown advised that on preliminary assessment there were a number of structural components that "may be under capacity". In particular, the base restraint to the cantilevered precast wall panels. In addition, Mr Brown considered the end connection to the inclined strut in grid B/2-3 "is vulnerable to instability when subject to severe seismic design actions". Mr Brown also considered that the "[i]nterconnected shear panels on grids 1 and A rely on site-welded embedments which are vulnerable to overload from temperature/shrinkage strains, with lintel panel being … particularly vulnerable."

REVISED DESIGNS

55. Mr Brown and Mr George have also provided further comment relating to the review and sign off, as well as the quality, of the revised designs (the 'check set' – undated and revisions S1 rev C, S2 rev B, S3 rev C, S4-S5 rev B, S6 rev C, S7 rev B, S8 rev C, S9 rev P1 dated either 3 March or 6 March 2003).

56. In relation to the 'check set' which Mr O'Connor says relates to changes on, or around, 3 March, Mr George said that they show "a very cursory review of the documents was carried out". Mr George noted that none of the issues he had noted in the original set of designs had been identified. Mr George stated: "I would have expected these items to have been raised during a review of the drawings."

57. Mr Brown also noted that his primary concern relating to the site welded connection of the precast concrete lintel panel was not identified on either the 'check set' or the revision set dated 3 or 6 March.

58. In relation to the adequacy of review of the designs Mr Brown stated:[7]

> "I would have expected Mr O'Connor, when reviewing these plans in the manner he did to have identified the risk inherent in this lintel panel support detail, and sought to modify it in the course of his 'high level' review of these drawings which ultimately conveyed his design to the builder."

59. See **Appendix A** and **B** for a full copy of their reports.

[7] This comment was made in relation to the 'check set', however, as discussed below we are unable to conclude whether Mr O'Connor reviewed the designs at that stage.

RESPONSE FROM MR O'CONNOR

60. In a statement to Engineering New Zealand dated 29 April 2017, Mr O'Connor stated that, in general, he considered that the engineering services for this building were provided in accordance with accepted standards. However, he stated that "there are some aspects that I would design differently in today's environment".

61. Mr O'Connor told Engineering New Zealand that KOA Ltd has carried out its own analysis of the building, including using a "3d equivalent static pushover analysis". Mr O'Connor provided copies of his calculations to support his view. Mr O'Connor stated:

> "In my view a transom should exist along the boundary wall to support the top of the precast panels. This omission I believe has come about due to a change in the precast panel sizes from being supported by the portals and foundation to being only supported at the foundation. Additionally, one purlin acting as a bracing strut needs the end connection bolts changing to grade 8.8 bolts – 4 bolts."

62. In relation to the GA Hughes & Associates Ltd report Mr O'Connor noted that the report is an initial assessment "and such reports can be unreliable". In relation to the Holmes Consulting report Mr O'Connor commented that "it is not clear that this is a Detailed Seismic Assessment".

63. Mr O'Connor stated that: "for many years roof bracing has been designed without concerns for large eccentricities and this has never been shown to be a problem in causing building failures."

64. In relation to the lack of fly bracing, Mr O'Connor submitted that his analysis indicates that no fly bracing is required and that "[t]he roof bracing has been shown to be adequate". Mr O'Connor has provided copies of some calculations carried out by KOA Ltd which he says support his views.

RESPONSE TO INVESTIGATING COMMITTEE'S PROVISIONAL DECISION

65. In response to the provisional decision Mr O'Connor noted that in its decision the Investigating Committee is considering his actions in relation to the signing of the PS1, but has not identified what accepted standards in relation to the use of PS1s are. Mr O'Connor referred to Engineering New Zealand's Practice Note 1 Guidelines on Producer Statements,[8] noting it states that "limited standardised practice has developed" in relation to the use of PS1s and that a producer statement is "not a product warranty or guarantee of compliance".

66. Further to this, Mr O'Connor reiterated his view that, based on the information he had available when signing the PS1, he had reasonable grounds to believe that the designs complied with the Building Code. Mr O'Connor stated:

> "In signing the PS1 I confirmed that I had a reasonable basis for thinking that, in my opinion, the designs complied with the Building Code. I did have a reasonable basis for thinking the designs complied with the Building Code. … [i]t was my practice to check aspects of the calculations in the designs, and to place weight on the fact the designs had been prepared and/or reviewed by an engineer with suitable experience and the person doing this work had 20 years experience as an engineer and the building is a relatively low risk structure smaller than most houses."

[8] See: www.engineeringnz.org/resources/practice-notes-and-guidelines/

67. Mr O'Connor noted that there was no requirement for him to "to exhaustively review the designs and calculations for the purpose of preparing the PS1".

68. In addition, Mr O'Connor expressed concern regarding the conclusion in Mr Brown's advice report of 30 April 2018, in which Mr Brown stated: "if the output PS1 for design … signed off by the CPEng/Principal is defective in terms of its statement … the author must accept responsibility arising from that." In relation to this statement Mr O'Connor stated:

> "This is not the inevitable consequence of a defective design, if the author of the producer statement took reasonable steps before signing it. A producer statement states that the author believes on reasonable grounds etc… Thus the issue is whether reasonable steps were taken, but in hindsight if those steps are found to reveal an error may have occurred that does not make the process unreasonable."

69. Mr O'Connor stated that the environment and standards have changed significantly since the Canterbury earthquakes. Mr O'Connor stated:

> "The environment we work in now as engineers and the standards expected have significantly changed because of the scrutiny applied to the profession since the Christchurch earthquakes. I believe if such scrutiny was applied to engineering work prior to this time, many practices and design details when judged by today's standards, would be discovered and deemed deficient."

70. Mr O'Connor reiterated his submission that many of the details that have been identified as being deficient in these cases were commonplace at the time the buildings were designed.

71. Mr O'Connor also noted he was not a Chartered Professional Engineer at the time he signed the PS1 for this building.

DISCUSSION

INVESTIGATING COMMITTEE'S ROLE AND THE LEGAL CONTEXT

72. Professional disciplinary processes primarily exist to protect the public, uphold professional standards, and maintain public confidence in the profession and its regulation. They do this by ensuring that members of the profession adhere to certain universal (or accepted) professional standards.[9]

73. Our role in this professional disciplinary process is to determine whether there are grounds to dismiss the matter under clause 8 of the Engineering New Zealand Disciplinary Regulations. If none of these grounds to dismiss the matter apply, then the complaint must be referred to a Disciplinary Committee.

74. In order to determine whether there are grounds to dismiss the matter, we have investigated whether Mr O'Connor provided engineering services relating to the engineering design work on the building at 408 Queen Street in accordance with accepted professional standards at the time the building was designed. We are not concerned with whether the building strictly complied with the building code or the building's current seismic rating – our assessment is whether Mr O'Connor acted reasonably at all stages of his involvement in the design of this building, including when he reviewed the design and signed the PS1.

[9] *Dentice v Valuers Registration Board* [1992] 1 NZLR 720 (HC).

75. The legal test we need to assess Mr O'Connor's actions against is whether he acted reasonably and in accordance with what a reasonable body of his peers would have done in the same situation (i.e. did he act in accordance with accepted standards).

76. If the evidence is that Mr O'Connor acted in accordance with accepted standards then we can dismiss the matter[10] If the evidence is that Mr O'Connor did not act in accordance with accepted standards then we need to assess how significant his departure from accepted standards was – if it was minor, we may dismiss the matter as insufficiently grave to warrant further investigation;[11] if it is more than minor, and no other ground for dismissal applies,[12] we are required to refer the matter to a disciplinary committee.

77. The issue we have specifically considered is whether in signing the PS1 Mr O'Connor was acting in accordance with accepted professional standards at the time the building was designed.

REVIEW AND SIGN OFF OF THE DESIGN BY WAY OF PS1

78. Mr O'Connor signed and dated the PS1 for this building on 24 February 2003. The PS1 refers to plans S1-S8a. The revision version is not recorded. The set of drawings held on the Masterton District Council property file are undated and recorded as being revision "P1". The calculations held on the Council file are dated 24 February 2003.

79. Mr O'Connor told the Investigating Committee it is evident from the KOA Ltd records that the PS1 was sent to Proarch Architects Ltd on 25 February. However, it appears from the information on the Council file that the plans were sent, together with the Building Consent application, on 24 February. Although the revision is not stated on the PS1, we are satisfied that it is more likely than not that the PS1 relates to the set of designs held on the Council file.

Adequacy of design

80. We accept Mr George's and Mr Brown's advice that the original designs held on the Council files did not meet accepted standards.

81. Both Mr George and Mr Brown identified significant concerns with the design of this building submitted to Masterton District Council as part of the building consent application. These concerns relate primarily to the lintel panel over the door on Grid 1, the lack of bracing detail and connection detailing.

82. Mr George advised that the panel over the door on Grid 1 is dangerous, and that the "brace details, lack of eaves ties, lack of fly braces, light vertical wall reinforcement, and panel type C fixings all look deficient".

[10] Clause 8(a) of the Engineering New Zealand Disciplinary Regulations, on the basis that there is no applicable ground of discipline under Rule 4 of the Engineering New Zealand Rules.

[11] Clause 8(c) of the Engineering New Zealand Disciplinary Regulations.

[12] There are other grounds in the Engineering New Zealand Disciplinary Regulations for dismissing a complaint, including: where the subject matter of the complaint is trivial; the complaint is frivolous or vexatious or is not made in good faith; where the person alleged to be aggrieved does not wish action to be taken or continued; where the complainant doesn't have a sufficient personal interest in the subject matter of the complaint; or an investigation is no longer practicable or desirable given the time elapsed since the matter giving rise to the complaint. In our view, given the nature of the Inquiry, we cannot apply any of these grounds to reasonably dismiss this matter.

83. Mr Brown also considered that the lintel panel on Grid 1 was significant, advising that this was a "[c]ritical defect". Mr Brown also raised concerns about the end connection of the inclined 125 SHS strut on Grid B, which he considered "may be susceptible to instability under severe seismic design action". He also considered the site welded embedments used to provide structure connection between adjacent shear panels on Grid 1 and Grid A "not to be sufficiently robust", and the foundation base/wall connection to be "marginal in terms of capacity".

84. We note Mr O'Connor's submission that, in his opinion, the design was reasonable and met relevant standards, although he did note "there are some aspects that I would design differently in today's environment". Mr O'Connor said that his assertion is supported by the reviews and checks carried out internally by KOA Ltd. Mr O'Connor provided copies of his calculations.

85. Furthermore, in response to the provisional decision, Mr O'Connor submitted that many of the details that have been identified as being deficient were commonplace at the time of the building's design and are the same as those used by Holmes Consulting in recent examples. Mr O'Connor stated:

> "The environment we work in now as engineers and the standards expected have significantly changed because of the scrutiny applied to the profession since the Christchurch earthquakes. I believe if such scrutiny was applied to engineering work prior to this time, many practices and design details when judged by today's standards, would be discovered and deemed deficient."

86. As noted above, our experts were asked to advise whether, in their opinion, the engineering designs were produced in accordance with accepted standards at the time and consistent with what a reasonable engineer, in the same circumstances, would likely have done. While the GA Hughes & Associates Ltd and Holmes Consulting reports focus on the current performance of the buildings and the issues identified relate to current standards, Mr George and Mr Brown's advice relate to the accepted standards at the time the buildings were designed – 2003 in the case of the building at 408 Queen Street.

87. We note that both the experts considered all the information provided to Engineering New Zealand by Mr O'Connor, including his submission of 29 April 2017, as well as a summary of the Investigating Committee's interview with Mr O'Connor on 26 March 2018.

88. Overall, we remain unconvinced by Mr O'Connor's submission. Guided by our experts, we confirm our view that the initial design for this building was seriously flawed, taking into account the year that the building was designed.

Should Mr O'Connor have signed the PS1?

89. By signing the PS1 Mr O'Connor was confirming his professional opinion that aspects of the building's design complied with the Building Code. Accordingly, Mr O'Connor took responsibility for the designs.

90. Therefore, the next question for us to consider is whether it was reasonable for Mr O'Connor to sign off the building's design by way of PS1, or whether he should have identified and responded to the issues identified by our experts, as noted above, before providing his sign off. In other words, the question for us is whether Mr O'Connor carried out his part in the review and sign off process in a careful and competent manner and in accordance with accepted standards.

91. The starting point for this assessment is to consider Mr O'Connor's role in the review and sign out process.

92. Mr O'Connor told Engineering New Zealand that, as per the usual quality assurance procedures in place at KOA Ltd at that time, he checked the designs prior to them being sent to Proarch Architects Ltd. He said that he was not directly involved in the design development of this building but would have carried out the final review prior to sign off of the designs, by way of PS1.

93. Mr O'Connor noted in his response to the provisional decision that there is no standard practice in relation to the use of PS1s and he refers to Engineering New Zealand's Practice Note 1 *Guidelines on Producer Statements* (the Practice Note) to support this. He also said that there was no requirement for him to "exhaustively review the designs and calculations for the purposes of preparing the PS1". Mr O'Connor submitted that based on his review and assurance provided by the senior engineer involved he had "a reasonable basis for thinking the designs complied with the Building Code".

94. We note that the Practice Note is referring to the use of PS1s by Chartered Professional Engineers operating under the Building Act 2004. Furthermore, we note that in 2003, the (now repealed) Building Act 1991 provided expressly that a council may accept a producer statement as a means of establishing compliance with the Building Code.[13] Accordingly, the Practice Note is not strictly applicable in this case. In our view, in 2003, there would have been a clear expectation from Mr O'Connor's peers that an engineer signing off a design by way of PS1 would take sufficient steps in their review of a design to satisfy themselves that they had reasonable grounds for signing the PS1.

95. We accept there was no requirement to undertake an "exhaustive" review himself. However, as noted above, in signing the PS1 Mr O'Connor was confirming his professional opinion that aspects of the building's design complied with the Building Code. Regardless of Mr O'Connor's trust in the experience and competence of the designer or the senior engineer who carried out the checks, as the person signing off the designs by way of a PS1, Mr O'Connor had a responsibility to carry out an adequate review himself, and not just rely on the reviews that may or may not have been carried out at other steps of the system. As noted by Mr Brown:[14]

> "In the final event, if the output PS1 for design … signed off by the CPEng/Principal is defective in terms of its statements (and irrespective of the reliance placed on verbal advice given by a Senior Engineer), the author must accept the responsibility arising from that. Put another way, the process that the CPEng/Principal used in reaching his/her judgement regarding the sufficiency of building design documentation is irrelevant if the statement is defective."

96. We note that in response to the provisional decision Mr O'Connor argued that if the author of a PS1 took reasonable steps to check the design met the relevant standards, if an error is later identified, "that does not make the process unreasonable".

97. We agree with Mr O'Connor that the issue is whether he took reasonable steps to check the designs. In our opinion, the evidence supports the view that he failed to do so in this case. In particular, Mr George stated that the "PS1 should not have been signed" for this design. Similarly, Mr Brown stated that the "CPEng/Principal sign off was seriously flawed in terms of [the site welded panel] detail".

[13] Section 33 of the Building Act 1991.

[14] See report dated 30 April 2018.

98. Overall, in considering the evidence as set out above, it is our view that in signing the PS1 for the building at 408 Queen Street in 2003 Mr O'Connor was not providing engineering services in accordance with accepted standards in the circumstances. We consider that, as the engineer signing off the designs by way of PS1 Mr O'Connor should have identified the issues as noted by our experts and, in the very least, taken additional steps in light of those issues to satisfy himself that the building met relevant standards before signing the PS1. That may have included, for example, querying this with the design engineer or senior reviewing engineer. There is no evidence that he did identify and respond to these issues.

99. Furthermore, in our view, a reasonable engineer would document any process of checking and clarifying aspects of a design at the sign-off stage, and included documentation to support their assertions.

100. We consider that a reasonable body of Mr O'Connor's peers would consider his actions in signing the PS1 (and without identifying and responding to the design issues which have been identified by our experts), was not consistent with accepted standards at the time. We regard this departure from accepted standards as more than minor. In these respects, we consider that there is an applicable ground of discipline in this case, and we do not consider that any of the grounds to dismiss this matter apply. Whether Mr O'Connor's actions are a breach of his professional and ethical obligations and whether they reach the threshold for professional discipline is a matter for a disciplinary committee.

101. We note Mr O'Connor's point that he was not a Chartered Professional Engineer at the time the design was issued. He was, however, a member of Engineering New Zealand (then known as IPENZ), and therefore subject to the organisation's Disciplinary Regulations. We confirm that our investigation has been carried out in accordance with the applicable Disciplinary Regulations.

REVISED PLANS

102. Mr O'Connor told the Investigating Committee that after the original plans were provided to Proarch Architects on 24 February 2003 (and after the PS1 was signed) they were changed significantly. In particular, Mr O'Connor said the building length was reduced by approximately two metres, and the panels adjusted accordingly. In addition, the layout was changed so the building had three equal bays and building services were added.

103. Mr O'Connor provided the Investigating Committee with a 'check set' of plans, which Mr O'Connor said he believes relate to changes made on or around 3 March. He also provided a further revised set of plans dated 3 or 6 March. Mr O'Connor said he never reviewed these revisions as per the normal review and sign out process. Mr O'Connor said that on review of the revised designs there is an obvious error relating to the transom and it appears that this error "slipped through the cracks" because his final check was never carried out.

104. Due to a lack of evidence, we are unable to make any conclusions regarding whether Mr O'Connor did or did not review the revised plans, although in light of his correspondence with Proarch Architects Ltd on 1 March, it is clear that he was aware the changes were being made.

105. Regardless, the revised plans are not held on the Masterton District Council property file for this property and there is no record of the building consent being amended. Furthermore, no investigation has been carried out on the existing building and it is unclear what building was ultimately built. Accordingly, without further analysis we are unable to make any conclusions about the existing building and whether it was built in accordance with the consented plans or the revised plans, as suggested by Mr O'Connor.

106. However, we note that our experts have significant concerns relating to the revised plans. Mr George advised that the design issues he noted on the original designs had not been picked up on the 'check set'. Mr George stated: "I would have expected these items to have been raised during a review of the drawings." Mr Brown also noted that neither the check set nor the 3 or 6 March revisions address the significant concerns relating to the lintel panel he noted on the original designs. Mr Brown stated:

> "I would have expected Mr O'Connor, when reviewing these plans in the manner he did to have identified the risk inherent in this lintel panel support detail, and sought to modify it in the course of his 'high level' review of the drawings which ultimately conveyed his design to the builder."

107. While, as noted above, we are able to conclude that Mr O'Connor did in fact review the revised plans, these comments still apply to the initial review Mr O'Connor apparently carried out prior to signing the PS1. In other words, Mr Brown's statement above supports our view that Mr O'Connor should have identified the issues with the original design when he signed the PS1.

108. These observations are concerning and, in our opinion, further confirm that Mr O'Connor should not have signed out the original design by way of PS1.

OTHER COMMENT

109. While beyond the scope of this investigation, on the basis of the information before us, we have a number of concerns about the development of this building. In particular, we are concerned that updated drawings and calculations do not appear to be held on the Council files, nor is there any record on the Council files that the building had been changed. As noted above, without further investigation we are unable to conclude what was ultimately built. If we accept that changes were made to the building after the building consent was issued, it is not clear whether these changes are not recorded anywhere because the Council file is incomplete. Alternatively, it is not clear whether the initial designs were the plans on which the building consent was granted, with the Council having never received the updated designs/plans.

CONCLUSION

110. Overall, we consider that it was inappropriate for Mr O'Connor to sign the PS1 in the circumstances described above and as set out in the information considered by us above.

111. In our opinion, Mr O'Connor should have identified the issues noted by our experts when reviewing the design for sign off and taken additional steps to reassure himself the designs met the relevant standards. We consider his failure to do so was more than a minor departure from accepted standards.

112. Accordingly, we consider that there are no grounds to reasonably dismiss the matter on the information collected to date and have decided to refer it to a disciplinary committee in accordance with clause 11(a) of the Disciplinary Regulations.

Andrew McMenamin CPEng CMEng
Chair of Investigating Committee

Stewart Hobbs CPEng FEngNZ IntPE(NZ)
Dr Sulo Shanmuganathan CPEng FEngNZ
Committee Members

INVESTIGATING COMMITTEE DECISION

OWN MOTION INQUIRY ABOUT KEVIN O'CONNOR — CNR DIXON AND CHURCH ST

For release

In accordance with:

Chartered Professional Engineers of New Zealand Act 2002
Chartered Professional Engineers of New Zealand Rules (No 2) 2002

Issued by
Andrew McMenamin CPEng CMEngNZ
Chair of Investigating Committee

Stewart Hobbs CPEng FEngNZ IntPE (NZ)
Dr Sulo Shanmuganathan CPEng FEngNZ
Committee Members

5 February 2019

engineering new zealand
Institute of Engineering Professionals

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BACKGROUND

1. In 2015, Engineering New Zealand[1] was made aware of structural engineering design concerns surrounding six buildings in Masterton owned by Masterton Trust Lands Trust (MTLT). The concerns were raised by a Chartered Professional Engineer (CPEng) engineer.

2. The concerns related to the structural integrity of the buildings. Because Engineering New Zealand has no jurisdiction over physical assets, Engineering New Zealand brought the concerns to the attention of the Ministry of Business, Innovation and Employment (MBIE), as the relevant regulatory authority.

3. The building, located on the corner of Dixon and Church Streets, was not one of the original six buildings identified. However, it was included in the six buildings ultimately selected by MBIE to assess further.

4. MBIE subsequently commissioned GA Hughes & Associates to carry out a structural review of the buildings, which identified concerns with the buildings and recommended a Detailed Seismic Assessment (DSA) be carried out. Following the receipt of this report, MTLT and MBIE commissioned a DSA to be carried out by Holmes Consulting, which identified concerns about the structural integrity of the six buildings. MTLT subsequently commenced remedial works on some of the buildings.

5. Based on the findings of these two reports, Engineering New Zealand decided that it needed to act on this information to determine if there is an issue with the engineering design of these buildings and, if so, what that means. It was decided that the best way to do this was by way of an own-motion inquiry pursuant to rule 55(1) of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002 (the Rules).

SCOPE OF THE INQUIRY

6. The scope of the Inquiry was to conduct an initial investigation into:

    "The circumstances relating to the design, design review and construction monitoring of six buildings in Masterton:

    - to assess whether the Chartered Professional Engineers involved have provided engineering services in accordance with accepted standards; and
    - to learn and advise on any engineering performance and practice improvements, if necessary."

7. Kevin O'Connor & Associates Ltd (KOA Ltd) was the engineering firm involved in the engineering design of all six of the buildings.

[1] On 1 October 2017 IPENZ changed its name to Engineering New Zealand. Accordingly, I will refer to Engineering New Zealand throughout this report.

8. The building on the corner of Dixon and Church Streets was designed and built in 2014. Kevin O'Connor CPEng[2] CMEngNZ[3] IntPE(NZ)[4] was identified as the engineer who reviewed and checked the design of the building and signed the original PS1.

9. The issue being considered with respect to Mr O'Connor is:

- Whether Chartered Professional Engineer Kevin O'Connor provided engineering services relating to the engineering design work on the building on the corner of Dixon and Church Streets in accordance with accepted standards.

INVESTIGATION

10. Following an initial investigation this matter was referred to an Investigating Committee for formal investigation. The Investigating Committee is:

Andrew McMenamin CPEng CMEngNZ (Chair)

Dr Sulo Shanmuganathan CPEng FEngNZ[5]

Stewart Hobbs CPEng CPEng FEngNZ IntPE(NZ)

11. Information collected by Engineering New Zealand for the purposes of this initial investigation included information from:

Kevin O'Connor	Respondent/engineer
Mr Y	Engineer and KOA employee
Kevin O'Connor & Associates Ltd	Engineering company
Masterton District Council	Local authority
Masterton Trust Lands Trust	Building owner
GA Hughes & Associates Ltd	Consulting structural engineering company
Holmes Consulting	Consulting structural engineering company

12. Independent expert advice was obtained from Barry Brown CPEng FEngNZ IntPE(NZ) (**Appendix A**) and Stuart George CPEng CMEngNZ IntPE(NZ) (**Appendix B**).

[2] Chartered Professional Engineer.

[3] Engineering New Zealand Chartered Member.

[4] A member of the New Zealand section of the International Professional Engineers.

[5] Engineering New Zealand Fellow

INFORMATION GATHERED

13. In 2013, Masterton Trust Lands Trust (MTLT) engaged Proarch Architects Ltd (Proarch) to develop a new building, located on the corner of Dixon and Church Streets, to be used as office space.

14. MTLT engaged Kevin O'Connor & Associates Ltd (KOA Ltd) to provide engineering design services for the development. While the contractual engagement was between MTLT and KOA Ltd, MTLT told Engineering New Zealand that Proarch acted as the project manager for the project and that all communications with KOA Ltd and the Masterton District Council went through Proarch.

BUILDING DESIGN

15. The building type is a single storey commercial/office building, with a footprint of two overlapping rectangles and an approximate floor area of 175m². The foundation condition comprises gravels (with allowable bearing of 300 kPa) with a 150mm thick concrete ground slab on grade, and edge thickening 200mm wide in the perimeter walls. The large pad foundations support cantilever precast concrete walls. The external fabric is a metal profile roof – monoslope – supported by steel purlins and lightweight wall cladding on timber, or steel framing with cantilever precast concrete walls.[6]

STRUCTURAL CONCEPT

16. Gravity roof loads are transferred from steel purlins to precast concrete walls or light timber framed walls.

17. The roof structure connects via a cross-braced diaphragm, formed from a series of overlapping light gauge steel cross-braces.

18. Lateral loads restraint is provided in the longitudinal and transverse directions through shear walls supported on large spread footing which act as cantilevers under face loads or shear wall under in-plane loads, or through numerous plasterboard lined timber framed shear walls.

ENGINEERING DESIGN

19. The engineering drawings describing the structural content of the designs are dated December 2013 and are initialled by the design engineer "Ms X", a KOA Ltd employee. Ms X became a CPEng on 7 September 2016.

20. Calculations and specifications, also completed by the design engineer, are dated November/December 2013.

21. Mr O'Connor told Engineering New Zealand that Ms X carried out the design work and that this was supervised and reviewed by one of KOA's senior engineers in the same office as Ms X. Mr Y, the CPEng engineer who signed the second PS1 dated 1 April 2014 relating to this building, told Engineering New Zealand that the senior engineer involved was Mr Z, who was the manager of the Palmerston North KOA Ltd office. Mr Y explained that the Palmerston North office was open-plan and that Ms X sat next to, or near, a number of senior engineers. He said that she could easily access assistance and supervision if needed.

[6] This description is taken from both Barry Brown and Stuart George's descriptions in their initial expert advice reports dated 23 May and 28 May 2017 respectively.

22. In relation to Mr Z's role in providing supervision of the design, Mr Ysaid that in accordance with the normal KOA Ltd review processes, Mr Z would have been available to assist Ms X while she was undertaking the design, and that Mr Z would have reviewed and checked the design once it was complete. This was confirmed by a signature at the bottom of the designs and on the original PS1.

23. Mr O'Connor told Engineering New Zealand that before signing the initial Producer Statement — Design (PS1) he conducted a brief review of the design and a check to ensure the design had been through a design check. Mr O'Connor stated:

    "My involvement prior to the PS1 being issued included a brief review of the design and a check to ensure the design had been through a design check as is evidenced by the initialled PS1 issued."

24. A copy of this document is not on the Masterton District Council property file, however, Mr O'Connor has provided Engineering New Zealand with a copy which confirms he signed the PS1 on 31 January 2014. The PS1, signed by Mr O'Connor, states:

    "On behalf of the Design Firm, and subject to:

    (I) Site verification of the following design assumptions safe ground bearing capacity of 300kPa (Gravel) …

    (II) All proprietary products meeting their performance specification requirements;

    I **believe on reasonable grounds** that a) the building, if constructed in accordance with drawings, specifications, and other documents provided or listed in the attached schedule, will comply with the relevant provisions of the Building code and that b), the persons who have undertaken the design have the necessary competency to do so." (emphasis in original)

BUILDING CONSENT

25. On 12 March 2014, Proarch submitted a building consent application to Masterton District Council on behalf of MTLT, which included the initial engineering designs (drawings SD1-SD2, BR01, S01-S07, SW1-SW5), as well as the specifications and calculations.

HIGH-LEVEL SCREENING REVIEW AND SUBSEQUENT PS1

26. On 17 March 2014, Masterton District Council wrote to Proarch advising that the structural specification and plans had been forwarded to the Council's contracted engineer to complete a "screening process" to ensure the designs complied with the Building Code, "in particular but not limited to, B1, B2, and G4".

27. Also on 17 March 2014, Masterton District Council engaged engineering firm Beca Group Limited (Beca) to carry out a "screen" of the "engineer's design for construction and compliance with B1 and B2 as the design relates to the building code". In addition, the request said, "please screen for compliance with G4 of the building code and NZS: 4219 for seismic restraint."

28. On 31 March 2014, Beca wrote to Masterton District Council advising that it had commenced a high-level screening review of the structural package, and requested information regarding the ground bearing capacity used for the structural design, noting that two differing values were referred to in the initial design documentation. In the letter to the Council, Beca stated:

"Please clarify what ground bearing capacity has been used for the structural design. The calculations make reference to qsafe = 300kPa and qult = 900kPa. Furthermore, Producer Statement makes note of 'safe ground bearing capacity of 100kPa'. Please confirm which value(s) is/are correct."

29. Also on 31 March 2014, Masterton District Council wrote to Proarch, notifying the firm of the screening review. The Council noted the process was a high-level screening and not a peer review for the issue of a producer statement PS2.

30. The Council forwarded Beca's request for information to Proarch, who in turn forwarded it to KOA Ltd. On 1 April 2014, Ms X responded on behalf of KOA Ltd and said:

"For the[…] building located [on the corner of Dixon and Church Streets] in Masterton; and in response to Masterton district council query – building consent application no. 140105 letter dated 31/03/14. Confirming assumed ground bearing capacity (>600mm depth) as follows:

- qsafe= 300kPa (Gravels)
- qult= 900kPa
- ϕqult= 720kPa (strength reduction factor ϕ=0.8 seismic)
- ϕqult= 450kPa (strength reduction factor ϕ=0.5 other load combination)"

31. An amended PS1 "with the correct safe bearing pressure" was provided.

32. The amended PS1, dated 1 April 2014, was signed by Mr Y CMEngNZ CPEng, and stated:

"On behalf of the Design Firm, and subject to:

(I) Site verification of the following design assumptions safe ground bearing capacity of 300kPa (Gravel) …

(II) All proprietary products meeting their performance specification requirements;

I believe on reasonable grounds that a) the building, if constructed in accordance with drawings, specifications, and other documents provided or listed in the attached schedule, will comply with the relevant provisions of the Building code and that b), the persons who have undertaken the design have the necessary competency to do so."

33. Mr Y only became involved in the project in March 2014 when the revised building consent documents were forwarded to him for review, following the Council's request for the geotechnical conditions to be stated on the PS1. Mr O'Connor was not available at that time, so Mr Y was engaged to sign the updated PS1 with this inclusion.

34. Mr Y told Engineering New Zealand that before signing the PS1 he reviewed the relevant documents and had a discussion with the design engineers. Taking into consideration that conversation, his understanding that the design had already been internally and externally reviewed, and that the updates he was being asked to sign off did not alter the original design or its intent, Mr Y signed the revised PS1.

35. On 1 April 2014, Proarch forwarded Ms X's letter, together with Mr Y's signed PS1 dated 1 April 2014, to Masterton District Council.

36. On 10 April 2014, Beca wrote to Masterton District Council advising the high-level screening of the structural engineer's design was complete. Beca confirmed it had received KOA Ltd's response to the query in its letter of 31 March 2014, and said:

> "[We] recommend that the structural design is accepted and appears to be compliant with part B1 and B2 (structural elements only) of the New Zealand Building Code. The following recommendations were made:
>
> 1. The design engineer is to monitor construction of elements of specific design to CM3[7] and provide a PS4 upon completion.
> 2. Soil parameter of 300kPa safe ground bearing capacity is to be verified on site."

BUILDING CONSENT ISSUED

37. A Masterton District Council 'Processing Checklist Commercial Building' dated 15 April 2014, in relation to the seismic restraint compliance with B1 of the Building Code, records that the PS1 was accepted as demonstrating B1 compliance.

38. On 16 April 2014, building consent was issued. No conditions are listed on the building consent document. However, an undated 'Building Consent Addenda' set out a list of conditions, including the following, which reflect Beca's recommendations:

> "1. The design engineer is to monitor construction of elements of specific design to CM3 and provide a PS4 upon completion. 2. Also, soil parameter of 300 KPa safe ground bearing capacity is to be verified on site."

CONSTRUCTION

39. Construction was carried out between April and September 2014.

40. In a letter dated 14 May 2014, from Proarch to Masterton District Council, responding to the Building Consent Addenda, Proarch said: "We reference the Building Consent Addenda … Refer attached site soil tests confirming bearing capacity of 300 kPa at 1100 below existing ground level. Footings are to be excavated to this depth, then backfilled with compacted metal to design levels."

41. The following inspections were carried out by the Council:

- Pre-wrap inspection (undated)
- 22 May 2014 – foundation inspection.
- 13 June 2014 – pre-pour/floor plumbing inspection
- 16 June 2014 – pre-slab inspection.
- 27 June 2014 – drainage
- 10 July 2014 – exterior cladding Stucco/EPS
- 15 July 2014 – preline-plumbing
- 16 July 2014 – pre-clad

[7] CM3 monitoring is defined as: "Review, to an extent agreed with the client, random samples of important work procedures, for compliance with the requirements of the plans and specifications and review important completed work prior to enclosure or on completion as appropriate."

- 25 July 2014 – prelining reinspection
- 11 September 2014 – final inspection, noted "Ok to issue CCC [Code Compliance Certificate]."

42. The Council inspection record states: "Engineer (KOA) has done soil test and inspected grid 1 yesterday and made arrangements with builders re continuation of inspections. Pour with his approval. …"[8]

43. The Council records also state: "Sighted plans, refer to details as per engineer's design (KOA-SO2) 062-500 … Pour on engineer's approval."

PS4 — CONSTRUCTION REVIEW

44. On 4 August 2014, a Producer Statement – Construction Review (PS4) was signed by Mr Y, confirming that part of the building work had been completed in accordance with the relevant requirements of the building consent and B1 of the Building Code.

CODE COMPLIANCE

45. A Code Compliance Certificate was issued on 11 September 2014.

SUBSEQUENT REVIEWS OF DESIGNS

46. On 4 August 2014, MTLT wrote to Proarch, advising it had become aware of engineering design issues relating to another project that KOA Ltd had provided engineering design for. MTLT requested advice as to whether similar issues may also apply to any of its buildings that KOA Ltd had been involved in. This letter was forwarded to KOA Ltd. On 23 September 2014, Mr O'Connor responded directly to MTLT saying he had carried out a desktop review of all the relevant buildings.[9] He said in his opinion, "the buildings … [have] been properly designed and detailed."

47. On 3 November 2014, MTLT responded that a review by Mr O'Connor or any current KOA Ltd employee did not provide the level of necessary comfort from an independent perspective. MTLT requested advice on how this could be achieved and whether it should consider requesting a peer review.

48. On 21 November 2014, Mr O'Connor responded that he understood KOA Ltd could be seen to have a financial interest, which could be perceived to impair the personal judgement of the reviewers, and recommended MTLT seek a detailed seismic assessment.

49. It does not appear that any further action was taken by MTLT at that time.

GA HUGHES & ASSOCIATES LTD

50. MBIE engaged GA Hughes & Associates Ltd (GA Hughes) to carry out a "High Level Structural Review" of the building, which was completed in June 2016.[10] The information considered in this review included the Masterton District Council property file, including the engineering calculations and drawings. A site visit and visual inspection of the building was also carried out. GA Hughes also undertook an Initial Evaluation Procedure (IEP) for the building, which is a quick assessment method based on factors such

[8] A Scala Penetrometer Test was carried out on 7 May 2014.

[9] This included the building on the corner of Dixon and Church Streets .

[10] The GA Hughes report is dated 21 June 2016.

as the buildings age, the type of materials used, and the construction type, used to identify if a building is potentially earthquake-prone.

51. In summary, GA Hughes identified the following structural vulnerabilities:

> "The use of face loaded walls for bracing together with timber framed walls is unusual and issues of displacement compatibility arise [especially] between the relatively flexible face loaded walls lined with [Gib] board.
>
> - Connections between walls and bracing should be subject to a detailed review.
> - The use of '[Drossbach]'[11] raises issues of construction quality. We are unaware what level (if any) of construction monitoring was carried out. The steel percentage in the lower part of the wall with 25 starters in the [Drossbach] at 150 centres appears to exceed the amount of reinforcing allowed by the concrete standard.
> - There are no fly braces and little restraint for the UB[12] & PFC[13] rafters and beams."

HOLMES CONSULTING

52. MBIE engaged Holmes Consulting to carry out a "Detailed Seismic Assessment" with a review of the structural capacity of vulnerabilities identified by GA Hughes. The review was completed in September 2016.[14] The review looked at load paths and configuration, as well as key detailing that may affect the seismic capacity of the building. The assessment involved review of documentation and calculations, where necessary. No visual inspection was undertaken. The assessment used the following approach:

> "Assess the building performance using the 2006 Assessment and Improvement of the Structural Performance of Buildings in Earthquakes (Red Book) and the draft for upcoming Seismic Assessment of Existing Buildings (Part C6)."

53. The review estimated the building's strength was 55-65% of the New Building Standard (NBS). It said:

> "[t]his result comes from the out-of-plane wall cantilever capacity. It is noted however that the roof bracing providing out-of-plane ties to the walls will reach their ultimate capacity at 30-35% of current code demand, however the precast walls have a base connection capable of sustaining out-of-plane moments, although it is possible that the foundations will rock at a similar level to the moment demands."

54. In relation to the metal strap bracing as shown on the plans, the review said "it is likely this bracing will yield under moderate earthquake loads, rendering it ineffective as a lateral load path". Therefore, the primary lateral load system was the out-of-plane bending of the precast concrete walls.

[11] Drossbach ducting is used in precast concrete connections.

[12] Universal beam.

[13] parallel flange channel.

[14] The Holmes Consulting report is dated 22 September 2016.

55. The assessment considered the seismic loads being applied in any direction and said:

> "precast concrete walls within the building have been analysed as nominally ductile with a strength reduction factor of 0.7. As a result, the 185 millimetre thick wall along the building's east elevation has an estimated strength of 60% of the strength of an equivalent new building."

56. Holmes Consulting said it was difficult to predict the behaviour of singly reinforced concrete walls under biaxial bending. It considered that in this building, walls subjected to biaxial bending would crack on an inclined plane through the wall's thickness.

> "As a result, when lateral loads on the wall reverse, and cracks formed under tension close, the potential for the wall becoming offset across an inclined crack and bar buckling could lead to wall instability. Potentially, this would compromise a critical vertical load path for the building and severely limit the concrete wall's ability to resist out-of-plane loads. As a result a ductility higher than $\mu_{=}$ 1.25 is not considered applicable in the assessment of the seismic capacity."

57. The review said that the timber-frame bracing was not likely to provide a reliable seismic load path in this case.

EXPERT ADVICE

58. Engineering New Zealand engaged two independent expert structural engineers — Stuart George and Barry Brown — to provide advice on this matter. They were asked to advise whether, in their opinion, the engineering designs were produced in accordance with accepted standards and consistent with what a reasonable engineer, in the same circumstances, would likely have done. They were not asked to undertake a seismic assessment of the building and neither have undertaken an evaluation of the per cent NBS. This is a key difference in the focus of the advice provided by Mr George and Mr Brown compared to the assessments undertaken by GA Hughes & Associates Ltd and Holmes Consulting – the Hughes and Holmes reports are instead concerned with the building's current performance.

59. The information Mr Brown and Mr George considered in forming their opinions included the Masterton District Council property file, Mr O'Connor's submissions, as well as the GA Hughes and Holmes Consulting reports.

60. In assessing whether the overall design met acceptable standards Mr George stated:

> "The definition of meeting acceptable standards is difficult to define. My best analogy would be marking a project at Engineering School, it is inevitable that [there] will be a wide variation in marks, and unlikely that anyone would score 100%. Even with the benefit of years of experience and an engineering registration process engineering knowledge and judgement varies."

61. Mr George gave the design an overall grade "C", highlighting concerns relating to the use of the precast concrete walls as the primary face load resisting element. He also considered the lateral load system looked to be inadequate.

62. Mr Brown also highlighted concerns regarding the lateral load system, but said:

> "Given the size of [the precast concrete wall panels] and the relative low seismic demand on them, I consider them to be adequate within themselves, although there may well be an issue with the effects of 'top of wall' rotation under face loads."

63. Mr Brown's primary concern was a "potential weakness in the beam to wall panel connection for the 9.3m long roof beam … supporting the roof". Mr Brown stated: "Lack of robustness in this end connection detail is a cause for concern."

64. Further to this, Mr Brown stated:

> "Associated with this primary defect is the lack of uniform lateral stiffness in the two (N, S) halves of the building and the potential for differential in-plane distortion (and possible associated damage) in the ceiling system under severe earthquake loads. Whilst the consequences of this may not impact life safety of the building occupiers, use of such a system does carry some risk."

65. Their reports are attached and marked Appendix A and B.

RESPONSE FROM MR O'CONNOR

RESPONSE TO COMPLAINT

66. In a statement to Engineering New Zealand, dated 26 April 2018, Mr O'Connor expressed surprise at Engineering New Zealand's inquiry which he said was directed to single storey buildings which had not, as far as he was aware, shown signs of failure and which he had no reason to believe were at risk of failing.

67. In response to whether he considered the engineering services provided in relation to the building on the corner of Dixon and Church Streets were in accordance with accepted standards, Mr O'Connor told Engineering New Zealand that the design was reviewed internally by KOA Ltd at the time of the initial design, and had subsequently been reviewed again by a senior KOA Ltd engineer who was a CPEng and MIStructE[15]. Mr Y had also reviewed the building.

68. Mr O'Connor said he conducted a brief review of the design and a check to ensure it had been through a design check, before signing the original PS1.

69. In relation to the GA Hughes report he noted this review was an IEP[16] and based on a document review and visual building inspection only. His view is it therefore cannot be relied on to be accurate.

70. Mr O'Connor also commented on the independent expert reports from Mr George and Mr Brown. In response to Mr George's statement that the "cantilever panels would be too flexible to protect non-structural elements", Mr O'Connor said that, in his opinion, "the cantilevered panels under face load the deflection are with[in] code limits".

[15] Chartered member of the Institution of Structural Engineers.

[16] Initial Evaluation Process (IEP) is a quick method of identifying if a building is earthquake prone.

71. In response to Mr Brown's concern regarding the robustness of the connection of the 9.3m beam, Mr O'Connor said the beam was carrying a metre of roof, was designed to deal with deflections, and was only a gravity-bearing element. He said he presumed Mr Brown's concern related to lateral deflection of the whole building and that these conditions could potentially prise the bolts from the panels. Mr O'Connor considered this unlikely given the connection capacity. He said:

> "The Ramset Epcon C6 fixings literature states the bolt fixing capacities per bolt for shear and tension to be albeit with a 125mm embedment not the 100mm shown on the drawings.
>
> ϕVus =31 Kn
>
> ϕNuc = 42 Kn
>
> Thus, each connection has four times this capacity. Thus, a single fixing would deal with the vertical loads and failure of all these fixings under earthquake loads seems unlikely."

72. Mr O'Connor also responded to Mr Brown's concern about the lack of uniform lateral stiffness between the North and South parts of the building and the potential for differential in-plane distortion in the ceiling system under severe earthquake loads. Mr O'Connor commented that he presumed this comment referred to the concrete panel stiffness (northern area) versus the timber rear (southern area) of the building. He said the two areas had a common roof and were supported laterally by precast panels in the northern section and timber braced walls in the southern area. Any deflections, in Mr O'Connor's opinion, would be small and unlikely to cause any concern in an SLS (Serviceability Limit State) event[17].

RESPONSE TO PROVISIONAL DECISION

73. In response to the provisional decision Mr O'Connor noted the findings of the experts differed, stating "this reflects how engineers can have differing opinions without either being wrong". Furthermore, in relation to the technical analysis by the experts, Mr O'Connor stated:

> "The expert reports also comment on the soil classification being class C soil in terms of NZS1170 but note using class D was acceptable.
>
> The building in question was designed using class D soil. Thus the loads the building was designed for are 25% higher than when using class C soil rather than class D. Thus the precast panels have been designed to resist elastic if one utilises class C soil.
>
> The experts note potential for differential movement between the timber braced area versus the majority area containing the precast panels appears to be of concern. The deflection of the panels at SLS loads are calculated to be 4.5 mm for face loading and would be much less for in plane loading and this reduces to 3.4 mm using class C soil. I would suggest the loads will transfer through the structure using secondary elements such as the roof etc at these level[s] of deflections without any damage. Loads above the SLS level may result in secondary damage but this is to be expected and is permitted under the code."

[17] The SLS is the level of stress or strain within the building below which there is a high expectation the building can continue as originally intended without repair.

74. In relation to his signing of the PS1, Mr O'Connor noted that at the time he signed the PS1 there was variation in practice with regards to their use and that the Investigating Committee has "no evidence before it of what accepted standards were in relation to PS1s at the time I signed the PS1." Mr O'Connor refers to Engineering New Zealand's Practice Note 1 Guidelines on Producer Statements,[18] noting it states that "limited standardised practice has developed" in relation to the use of PS1s and that a producer statement is "not a product warranty or guarantee of compliance".

75. Further to this, Mr O'Connor reiterated his view that, based on the information he had available when signing the PS1, he had reasonable grounds to believe that the designs complied with the Building Code. Mr O'Connor stated:

> "In signing the PS1 I confirmed that I had a reasonable basis for thinking that, in my opinion, the designs complied with the Building Code. I did have a reasonable basis for thinking the designs complied with the Building Code. … [i]t was my practice to check aspects of the calculations in the designs, and to place weight on the fact the designs had been prepared and/or reviewed by an engineer with suitable experience and the person doing this work had 20 years' experience as an engineer and the building is a relatively low risk structure, small[er] than most houses."

76. Mr O'Connor noted that there was no requirement for him to "to exhaustively review the designs and calculations for the purpose of preparing the PS1".

DISCUSSION

INVESTIGATING COMMITTEE'S ROLE AND THE LEGAL CONTEXT

77. Professional disciplinary processes primarily exist to protect the public, uphold professional standards, and maintain public confidence in the profession and its regulation. They do this by ensuring that members of the profession adhere to certain universal (or accepted) professional standards.[19]

78. Our role in this professional disciplinary process is to determine whether there are grounds to dismiss the complaint in rule 57 of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002. If none of these grounds to dismiss the complaint apply, then the complaint must be referred to a Disciplinary Committee in accordance with rule 60(a) of the Rules.

79. In order to determine whether there are grounds to dismiss the complaint, we have investigated whether Mr O'Connor provided engineering services relating to the engineering design work on the building on corner of Dixon and Church Streets in accordance with accepted professional standards at the time the building was designed. We are not concerned with whether the building strictly complied with the building code or the building's current seismic rating – our assessment is whether Mr O'Connor acted reasonably at all stages of his involvement in the design of this building, including when he reviewed the design and signed the PS1.

80. The legal test we need to assess Mr O'Connor's actions against is whether he acted reasonably and in accordance with what a reasonable body of his peers would have done in the same situation (i.e. did he act in accordance with accepted standards).

[18] See: www.engineering nz.org/resources/practice-notes-and-guidelines

[19] *Dentice v Valuers Registration Board* [1992] 1 NZLR 720 (HC).

81. If the evidence is that Mr O'Connor acted in accordance with accepted standards then we can dismiss the matter.[20] If the evidence is that Mr O'Connor did not act in accordance with accepted standards then we need to assess how significant his departure from accepted standards was – if it was minor, we may dismiss the matter as insufficiently grave to warrant further investigation;[21] if it is more than minor, and no other ground for dismissal applies,[22] we are required to refer the matter to a disciplinary committee.

82. The issues we have specifically considered in this case is whether in signing the PS1 Mr O'Connor was acting in accordance with accepted professional standards at the time the building was designed.

REVIEW AND SIGN OFF OF THE DESIGN BY WAY OF PS1

Adequacy of the design

83. In his statement dated 26 April 2018, Mr O'Connor submitted that the building complied with accepted design standards and is safe, and that this has been confirmed through various reviews conducted by KOA Ltd. Mr O'Connor did not provide us with copies of these reviews.

84. In contrast, both Mr George and Mr Brown identified concerns with the engineering design of this building, primarily relating to the load paths and wall panel/foundation slab joint connections. Mr Brown described the design as a "poorly developed structural concept". He stated:

> "The most serious primary defect is the potential for overload in the end connection of the long span beam on the north side, which effectively ties the two cantilevered walls together. Lack of robustness on this end connection detail is a cause for concern."

85. Mr George stated: "Cantilever panels would be too flexible to protect non-structural element[s]." He also advised that the interaction with the timber walls were unclear, the connection at the top of the braced walls were unclear and the lumberlock straps would be inadequate to act as bracing paths.

86. While we note Mr O'Connor's submission in response to our provisional decision, we do not consider he has provided us with any new information. Accordingly, guided by our experts, we are concerned that there are deficiencies with the engineering design of this building. We believe these deficiencies indicate the engineering design was not of an adequate standard.

Should Mr O'Connor have signed the PS1?

87. Chartered professional engineers use PS1s to confirm their professional opinion that aspects of a building's design comply with the Building Code. Their intent is to signal to a building consent authority (BCA) that certain design work has been done (or overseen/supervised) by a practitioner who is competent to perform the defined work.[23]

[20] Rule 57(a) of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002, on the basis that there is no applicable ground of discipline under section 21(1)(a) to (d) of the Act.

[21] Rule 57(ba) of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002.

[22] There are other grounds in the legislation for dismissing a complaint, including: where the subject matter of the complaint is trivial; the complaint is frivolous or vexatious or is not made in good faith; where the person alleged to be aggrieved does not wish action to be taken or continued; where the complainant doesn't have a sufficient personal interest in the subject matter of the complaint; or an investigation is no longer practicable or desirable given the time elapsed since the matter giving rise to the complaint (Rule 57 of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002. In our view, given the nature of the Inquiry, we cannot apply any of these grounds to reasonably dismiss this matter.

[23] Engineering New Zealand Practice Note 1 *Guidelines on Producer Statements* (January 2014), section 3.1.

88. Although they have no legal status under the Building Act 2004, PS1s are intended to provide BCAs with information to establish that there are reasonable grounds for the issue of a building consent. When used properly, they give BCAs confidence that certain building work will be constructed to meet the Building Code.

89. Chartered professional engineers should be aware that BCAs are likely to rely on producer statements to some extent, and therefore should be mindful that responsibilities and potential liabilities may arise from signing them.[24]

90. In signing the PS1 for the building on the corner of Dixon and Church Streets, Mr O'Connor was confirming his professional opinion that aspects of the building's design complied with the Building Code. Accordingly, by signing the PS1 Mr O'Connor assumed responsibility for the building's design.

91. Therefore, the question for us to consider is whether it was reasonable for Mr O'Connor to sign off the building's design by way of PS1, or whether he should have identified and responded to the issues identified by our experts, as noted above, before providing his sign off. In other words, the question for us is whether Mr O'Connor carried out his part in the review and sign off process in a careful and competent manner and in accordance with what a reasonable body of his peers would have done in the same situation.

92. The starting point for this assessment is to consider Mr O'Connor's role in the review and sign off process.

93. Mr O'Connor told the Investigating Committee that in signing off the building he would have carried out a quick check of the calculations and a high-level review of the designs, but said he was guided by the senior engineer involved (Mr Z) carrying out their structural check of the designs. Mr O'Connor said that the senior engineer in this case was very experienced. Mr O'Connor told the Investigating Committee that Mr Zr had initialled each of the drawings indicating that he had completed his in-depth review of the drawings.

94. Mr O'Connor noted in his response to the provisional opinion that there is no standard practice in relation to the use of PS1s and he refers to Engineering New Zealand's Practice Note 1 *Guidelines on Producer Statements* (the Practice Note) to support this. He also said that there was no requirement for him to "exhaustively review the designs and calculations for the purposes of preparing the PS1." Mr O'Connor submitted that based on his review and assurance provided by the senior engineer involved he had "a reasonable basis for thinking the designs complied with the Building Code".

95. The part of the Practice Note that Mr O'Connor refers to in support of his argument about standard practice states: "By the same token there is no provision for BCAs to require a producer statement, as of right. As a result of their non-mandatory status, limited standardised practice has developed. There is currently such a wide variation in the way BCAs accept and/or rely on producer statements that there is a resultant degree of confusion, frustration and inefficiency amongst practitioners and BCAs." We interpret the Practice Note to be saying that the lack of standardised practice related to the extent by which producer statements are used by BCAs; not the extent of engineering practice with regard to the level of review a chartered professional engineer should carry out before signing off a PS1 – which is the issue in this case. We consider that, in 2014, there would be a clear expectation from

[24] Ibid.

Mr O'Connor's peers that a chartered professional engineer signing off a design by way of PS1 would take sufficient steps in their review of a design to satisfy themselves that they had reasonable grounds for signing the PS1.

96. We accept that there was no requirement to undertake an "exhaustive" review himself. However, as noted above, in signing the PS1 Mr O'Connor was stating his professional opinion based on reasonable grounds that the building's design complied with the Building Code. Regardless of Mr O'Connor's trust in the experience and competence of the designer or senior supervising engineer, as a CPEng engineer and the person signing out the designs, Mr O'Connor had a responsibility to carry out an adequate review himself, and not just rely on the reviews that may or may not have been carried out at other steps of the system. As noted by Mr Brown:

> "if the output PS1 for design … signed off by the CPEng/Principal is defective in terms of its statements (and irrespective of the reliance placed on verbal advice given by a Senior Engineer), the author must accept the responsibility arising from that. Put another way, the process that the CPEng/Principal used in reaching his/her judgement regarding the sufficiency of building design documentation is irrelevant if the statement is defective."

97. Therefore, we must consider whether Mr O'Connor's actions in signing the PS1 were reasonable in the circumstances, based on the level of review he carried out, and that expected of a reasonable engineer in the same circumstances.

98. Mr George considered that the majority of the issues he identified should have been checked at the sign off stage of the review process. There is no evidence that they were.

99. Mr Brown noted that the structural calculations and drawings are both dated December 2013 which suggests that drafting and structural verification were carried out together. Mr Brown commented:

> "My suspicion is that this building was conceptualised by a design draftsperson, with engineering input being overlaid in an ad hoc manner after the primary design decision had been made by the drafter."

100. Mr Brown considered that the design was a "poorly developed design concept".

101. Overall, when weighing up the advice from the experts, it is our opinion that Mr O'Connor did not act in accordance with accepted standards in signing off this design by way of a PS1. We consider that, as the CPEng engineer signing off the design, and having carried out the level of review described, Mr O'Connor should have identified the issues as noted by our experts and, in the very least, taken additional steps in light of those issues to satisfy himself that the building met relevant standards before signing the PS1. That may have included, for example querying this with the design engineer or senior reviewing engineer. There is no evidence that he did identify and respond to those issues.

102. Further, in our view, a reasonable engineer would document any process of checking and clarifying aspects of a design at the sign off stage and included documentation to support their assertions. It appears that, at best, Mr O'Connor carried out a cursory review of the designs and calculations for this building before signing it off by way of PS1.

103. In our opinion, a reasonable body of Mr O'Connor's peers would likely consider that his actions in signing the PS1 (and without identifying and responding to the issues identified by our experts), was not consistent with accepted practice at the time, particularly given the year that this was designed. In these respects, we consider that there is an applicable ground of discipline in this case, and we do not consider that any of the grounds to dismiss this matter apply. Whether Mr O'Connor's actions are a breach of his professional and ethical obligations and reach the threshold for professional discipline is a matter for a disciplinary committee.

CONCLUSION

104. Overall, we consider that it was inappropriate for Mr O'Connor to sign the PS1 in the circumstances described above and as set out in the information considered by us above. In our opinion, Mr O'Connor should have identified the issues noted by our experts when reviewing the design for sign off and taken additional steps to reassure himself the designs met the relevant standards before signing the PS1. We consider this was more than a minor departure from accepted standards.

105. Accordingly, we do not consider that there are grounds to reasonably dismiss the matter on the information collected to date and have decided to refer it to a disciplinary committee in accordance with rule 60(a) of the Chartered Professional Engineers of New Zealand Rules (No 2) 2002.

Andrew McMenamin CPEng CMEngNZ
Chair of Investigating Committee

Stewart Hobbs CPEng FEngNZ IntPE(NZ)
Dr Sulo Shanmuganathan CPEng FEngNZ
Committee Members

APPENDIX A FOR DOCUMENTS B1-B5

EXPERT ADVICE REPORT FROM BARRY BROWN (REDACTED)

Fraser Thomas

ENGINEERS · RESOURCE MANAGERS · SURVEYORS

23 May 2017

Project No. 49158

General Manager - Professional Standards
IPENZ / Engineers New Zealand
PO Box 12145
Thorndon
WELLINGTON 6144

Attention Mr Brett Williams

Dear Brett

STRUCTURAL COMPLIANCE REVIEW - SIX MASTERTON BUILDINGS
- EXPERT ADVICE FOR IPENZ OWN MOTION INQUIRY

I refer to your letter of engagement with regard to the above matter dated 10/4/17 plus separate correspondence received from IPENZ Investigations Lead ([redacted]) who briefed me in Auckland on 8/3/17 (Investigator's letter dated 8/3/17 also refers).

I have completed the first phase of the investigation of the design documents (plans, specifications, calculations, etc) of the buildings in question and set out via this letter, together with the information contained in the attached application, my preliminary opinion on the questions raised regarding the sufficiency of the designs in question and thereby on the actions of those CPEng registered practitioners signing off on the design.

1.0 BACKGROUND

1.1 The subject buildings are all located in Masterton and all have the same owner, viz Masterton Lands Trust, with the engineering work involved being carried out through the same consulting firm, viz Kevin O'Connor Associates (KOA), and (for the purposes of this review) are described as follows :

1.2 Scope of Review

(1) FTL Ref	(2) Building Name	(3) Address	(4) PS1/PS4 Author	(5) Building Consent Issue Date	6) Preliminary Compliance Assessment (Appendix B)
49158/1	[redacted]	408 (390) Queen Street	Kevin O'Connor (24/2/03)	12/3/03	B1
49158/2	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
49158/3	[redacted]	cnr Dixon & Church Streets	Kevin O'Connor (6/10/06)	14/12/06	B3
49158/4	[redacted]	57-65 Dixon Street	Unknown (28/10/10)	14/1/11	B4
49158/5	[redacted]	196-120 Queen St	Kevin O'Connor (21/6/11)	26/7/11	B5
49158/6	[redacted]	57-61 Dixon Street	Kevin O'Connor (unknown) Mr Y [redacted] (1/4/14)	16/4/14	B6

1.3 A summary of key information provided by IPENZ at the initial briefing on 8/3/17 assisted me considerably in reviewing the large amount of design and construction records provided in relation to each building.

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AUCKLAND 2161, NEW ZEALAND
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www.fraserthomas.co.nz

2.0 METHODOLOGY

2.1 Given the open-ended nature of my briefing from IPENZ, my review has been necessarily broad in scope, having the objective of identifying specific items of non-compliance using "standards of the day" benchmarks which applied at the date of design/consenting.

2.2 Given the above, plus the time/budget limitations imposed on the review by IPENZ, once I received the relevant design documents, I determined that I would adopt the following approach in undertaking the work.

2.2.1 Commence with a review of plans and specifications for the buildings in question in order to :

(a) identify the load paths within the load resisting systems for :

(i) gravity loads, viz dead, superimposed, and wind verticality loads; and
(ii) lateral loads in each orthogonal direction, viz wind, earthquake, etc

- based on my experience of the design process normally used with this type of building; and

(b) for the lateral force resisting system, include consideration of the constructed form, and in particular the ductility (robustness) of the detailing of the primary structure as shown in the design documents, in terms of the operative verification methodology applicable at the date of consenting.

2.2.2 Having completed this, I sought to identify potential gaps (or weaknesses) in the design where :

(a) the load paths were unclear, complex, or potentially incomplete; and

(b) the details presented appeared undersize or insufficiently robust

- with the results of this evaluation being parked for separate (and later) consideration.

2.2.3 Finally, the design calculations presented within the building consent documentation were reviewed to clarify the designers' approach concerning the various aspects of the review previously considered.

2.2.4 The output of this information is recorded in summary form in Appendix B, with marked up plans and explanatory notes on the methodology used by designers of the day are included in Appendix C for further reference where appropriate.

2.3 My focus in preparing the output of this first stage review has been on the building as described by the design documents rather than the building as actually constructed, because I have not (as yet) been able to view the finished building.

2.4 Because the subject buildings are of similar type, viz "light commercial-industrial" buildings constructed using standardised "precast and tilted up" concrete wall components which are in part stabilised by a structural steel superstructure, it is acknowledged that this review has needed to consider a number of standard design approaches that have been developed by the industry over time.

2.5 During this period, the knowledge of design practitioners in the industry has improved, and some relevant design standards, eg Concrete Structures Standard NZS 3101, have been revised.

2.6 In preparing this report, I have placed considerable emphasis on :

(a) the relevant loading and materials design standards in use for achieving compliance with clause B1 of the Building Code; and

(b) the "state of knowledge" prevalent amongst the design profession relating to prediction of structural performance within industrial buildings of this type

- at the time the building consent was granted.

2.7 Whilst references are made to eg SESOC Interim Practice Note (reference R10 in Appendix C1), I have quite deliberately not sought to include unreasonably the lessons learned from the Canterbury earthquake experience, instead taking the view that the obligation of the practitioner is to both be informed by such events and respond to regulatory change, but not to necessarily lead such change without a consensus endorsement.

3.0 PRELIMINARY REPORT ON BUILDINGS' DESIGN COMPLIANCE

3.1 The results of our compliance review for each building is included within Appendix B, indexed as per Table 1/column 6.

3.2 Each building report within Appendix B is formatted under the following headings :

(a) Name, address, and designer (CP Engineer) of the building

(b) Date of Building Consent (BC) and Code Compliance Certificate (CCC)

(c) Relevant design documentation, viz drawing list, plus structural calculations

(d) Description of building in terms of how it works, and in particular how it resists lateral, eg wind, seismic, loads, including the relevant design, eg loading/materials, standards to be complied with

(e) Areas/items where non-compliances have been identified and/or suspected (these to be confirmed or deleted after further work and/or discussions with other structural engineers who have reviewed the same documents)

3.3 Concurrently with the above, I have undertaken a review of particular aspects of tilt up concrete construction which (I note) had been the subject of widespread industry research over the period prior to and during the times these buildings were designed. Reference is made to this within the various summaries included in Appendix C "Supplementary Information" under the following general headings.

(a) Appendix C1 : Various industry references on eg Tilt Up Construction

(b) Appendix C2 : Requirements of Loading Standards, eg Seismic Actions (NZS 4203:1992 Part 4, and NZS 1170.5 Parts 5 and 8)

(c) Appendix C3 : Indicative Information on Subsoil Classification for the subject sites

(d) Appendix C4 : Information on Proprietary Inserts for Reinforcement Anchorage in Precast Concrete Panels

(e) Appendix C5 : Calculations for Representative Structural Elements

NB : In the first issue of this report, I have provided the headings for Appendix C but have not in all cases included the contents of the representative sub-appendices, except for some representative calculations for TCM embedments on two of the buildings, viz 49158/1 (408 Queen St) and 49158/4 57-65 (Lot 9) Dixon Street).

3.4 Building Compliance As-Constructed

(a) This report has been prepared based on the design documentation made available to me.

(b) I have not, at the time of this reporting, undertaken a visit to Masterton to inspect the subject buildings.

(c) The need to undertake such a visit is a matter for future discussion with IPENZ/Lead Investigator and other reviewers.

4.0 GENERAL SUMMARY OF FINDINGS

4.1 The following information, included within Appendix B, is brought forward into this summary.

FTL Ref	Building Name	PS1/PS4 Author (date)	Relevant Design Standards	Preliminary Compliance Assessment (Appendix B)	General Findings (provisional)				Remarks / Notes
					None	Minor	Significant	Major	
49158/1*	408 Queen St [redacted]	Kevin O'Connor (24/2/03)	NZS 4203:1992 NZS 3101:1995 incl Amend 1	B1			✓		Refer Notes (2), (3), (4)
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]		✓			[redacted]
49158/3	57-65 Dixon St	Kevin O'Connor (6/10/06)	NZS 4203:1992 NZS 3101:1995 incl Amend 1	B3			✓		Refer Notes
49158/4*	57-65 (Lot 9) Dixon St	Unknown (28/10/10)	NZS 1170.5:2004 NZS 3101:2006	B4			✓		Refer Notes (2), (3), (4)
49158/5	196-120 Queen St	Kevin O'Connor (21/6/11)	NZS 1170.5:2004 NZS 3101:2006	B5			✓		Refer Notes (2), (3)
49158/6	Cnr Dixon and Church St	Kevin O'Connor (unknown) Mr Y (1/4/14)	NZS 1170.5:2004 NZS 3101:2006	B6			✓		Refer Notes (2), (5)

Notes :

(1) The content of this summary is indicative, and reference should be made to the full draft report within each section of Appendix B for detail.

(2) Design requirements specified within cited B1/VM1 verification methods, eg NZS 3101:1995, are deemed to specify a performance standard which the structural design is intended to achieve. The designer is free to choose alternative means of satisfying these performance requirements within the framework of the relevant loadings standard. In several instances, I have found it necessary to consider such alternative means of achieving compliance in particular cases in this review, as noted in the remarks column.

(3) Significant rigor defect identified relates to sufficiency of the design for the TCM connections to the vertical interface between cantilevering concrete wall and stabilising foundation.

(4) Representative calculations have been included within sub-Appendix C5 for two buildings denoted "*". These calculations are intended to indicate the accepted design methodology within the period during which they were designed.

(5) Design documentation not sufficiently complete for me to be confident that findings are accurate, and site inspection will be required to confirm as-built details.

5.0 LIMITATIONS

5.1 This review of the six subject buildings has been undertaken by me following the methodology agreed with IPENZ/Investigator at our briefing meeting on Wednesday 8/3/17.

5.2 Given the scope of this assignment, the work on the review has been carried out on a high level basis and within budget/time limitations which IPENZ has imposed. Consequently, I do not warrant that I have followed any externally prescribed review methodology, such as that outlined in Practise Note 02 "Peer Review - Reviewing the work of another Engineer" dated June 2003 as previously issued by IPENZ/ACENZ, in order to arrive at the preliminary conclusions I have recorded in this report.

5.3 In preparing this report, my review has been undertaken separately from that of the other reviewer engaged by IPENZ for this purpose, viz Mr Stuart George, except for :

(a) our joint attendance at the initial briefing with IPENZ on Wednesday 8/3/17;

(b) our sharing of information relating to applicable industry standards relating to the type of building construction concerned; and

(c) our meeting together on Tuesday 28/3/17 in order to share information on preliminary findings, useful reference information sources, and to develop a review methodology going forward.

NB : Record minutes of this meeting are held on file for reference if required by IPENZ.

5.4 My understanding from the initial briefing by IPENZ on 8/3/17 is that, following my delivery of this draft report to IPENZ, I will be given the opportunity to :

(a) discuss my findings with the other reviewer, viz BGT Structures/Stuart George;

(b) consider the contents of compliance review reports prepared by eg MBIE and others with regard to the same buildings; and

(c) discuss my findings with the KOA designer concerned.

6.0 CONCLUSION

6.1 This report represents the output of my preliminary review of the design of 6No light industrial/commercial buildings undertaken by KOA for the Masterton Land Trust over the 10-year period 2004 through 2014.

6.2 I trust these findings provide a basis for an appropriate consideration of the matters at issue by IPENZ investigation team, and further discussion with other parties involved.

6.3 I am available for whatever further discussion is required to clarify my findings.

Yours sincerely
FRASER THOMAS LIMITED

B J BROWN CPEng 43778
Principal - Structural Engineering

Appendices

A IPENZ LETTER OF ENGAGEMENT
- A1 IPENZ / Brett Williams letter dated 10/4/17
- A2 IPENZ / Investigator letter dated 8/3/17

B PRELIMINARY FINDINGS ON THE SIX MASTERTON BUILDINGS
- B1 408 Queen St
- B2 [redacted]
- B3 57-65 Dixon St
- B4 57-65 (Lot 9) Dixon St
- B5 196-120 Dixon St
- B6 Cnr Dixon and Church St

C SUPPORTING INFORMATION
- C1 Various industry references on eg Tilt Up Construction
- C2 Requirements of relevant Loading Standards, eg Seismic Actions (NZS 4203:1992 Part 4, and NZS 1170.5 Parts 5 and 8)
- C3 Information on Subsoil Classification for the subject sites
- C4 Indicative Information on Proprietary Inserts for Reinforcement Anchorage in Precast Concrete Panels
- C5 Basis of Design and Calculations for Representative Structural Elements

APPENDIX A

IPENZ LETTER OF ENGAGEMENT

A1 IPENZ / Brett Williams letter dated 10/4/17

REGISTRATION AUTHORITY
FOR CHARTERED PROFESSIONAL ENGINEERS

10 April 2017

Barry Brown
Fraser Thomas
By Email: **bbrown@ftl.co.nz**

Dear Mr Brown

RE: Letter of Engagement — Expert advice for IPENZ Own-Motion Inquiry

Thank you for agreeing to provide expert advice to IPENZ for the purposes of IPENZ's Own-Motion Inquiry into the engineering design of six buildings located in Masterton.

This letter sets out the terms of our engagement with you (Fraser Thomas).

The engagement

You will provide expert advice for IPENZ's Own-Motion Inquiry into the circumstances around the design of six buildings in Masterton. The purpose of your advice is to inform IPENZ in its decision-making as to whether the actions of any individual Chartered Professional Engineers require further consideration by IPENZ in accordance with its complaints and disciplinary process.

The process that you have agreed to follow for providing your advice was set out in Investigations Lead [redacted] email to you on 16 March 2017. This may be subject to change, with your agreement.

Fees

[redacted].

[redacted]:

- [redacted]
- [redacted]

Please invoice IPENZ within 30 days of any travel undertaken.

What we expect from you in delivering the Services

IPENZ expects the following standards from its experts:

- All information obtained, including information relating to IPENZ's work and projects, should be kept private and confidential.
- All documents provided for the purposes of your review should be kept completely confidential and returned to IPENZ at the completion of the review.
- Declare if you have a potential or actual conflict of interest, or if a potential or actual conflict of interest arises.
- Your advice should apply and reference any relevant professional or ethical standards and guidelines.
- Your advice should be limited to matters within your area of expertise.
- Direct any media queries or queries from the public about this Inquiry to IPENZ.

How we will support you to provide your advice

IPENZ will:

- Provide you with all the relevant materials.
- Provide you with a brief of the advice we are seeking.
- Gather any additional information that you need to provide your advice.
- Coordinate any travel necessary for you to provide your advice.

Dispute resolution

If a dispute arises between us about the nature of your engagement, or your or our responsibilities under this agreement then we will work together to resolve those. If we are not able to resolve the dispute, the dispute will be forwarded to the Chief Executive of IPENZ to resolve.

Complaints

In a situation where IPENZ receives a complaint about you in relation to advice that you have provided to IPENZ, this will be considered within the context of the complaint being investigated i.e, it will be considered as a response to the advice report rather than as a separate complaint against the advisor.

If a complaint is received about you on an unrelated matter this will be managed in accordance with IPENZ's standard complaints resolution process.

Indemnity

IPENZ agrees to indemnify, and agrees to keep indemnified, you from and against all actions, suits, proceedings, claims and demands whatsoever made or brought against you, or the Institution, by any third party in respect of, or arising out of the performance by you of your obligations hereunder, other than those arising out of gross negligence, wilful neglect, default or misconduct on your part.

B J Brown
ref Letter of
Engagement dated
10/4/2017

Letter of Engagement

Please contact Ms Byers if you need to discuss any part of this Letter of Engagement, otherwise please sign and return by email by 18 April 2017. Thank you for assisting us in this matter.

Yours sincerely

Brett Williams
General Manager — Professional Standards

I accept the terms as set out in this letter of agreement.

BARRY BROWN
c/ FRASER THOMAS LTD

Name

Signature

23/4/2017

Date

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A2 IPENZ / Investigator letter dated 8/3/17

8 March 2017

REGISTRATION AUTHORITY
FOR CHARTERED PROFESSIONAL ENGINEERS

Barry Brown

Delivered by hand

Dear Mr Brown

Re: **Masterton Own-Motion Inquiry**
Our ref: **410**

Thank you for agreeing to provide expert advice to IPENZ. IPENZ is seeking your opinion about the adequacy and appropriateness of the engineering designs and calculations in relation to six buildings in Masterton.

Own-Motion Inquiry
As advised, IPENZ has commenced an own-motion inquiry into the circumstances surrounding the engineering of six buildings in Masterton. This inquiry was prompted by some engineering assessments of these buildings which raise questions about their structural integrity. I **enclose** a copy of the Inquiry's Terms of Reference for your information.

Process
IPENZ is currently in the process of collecting and considering information about the design of these buildings. As part of its consideration IPENZ has decided that it would be appropriate to obtain independent expert advice on the matter. Initially this would involve providing advice on whether the engineering designs were produced in accordance with accepted standards and consistent with what a reasonable engineer, in the same circumstances, would likely have done. This advice will help guide IPENZ on what, if any, next steps may be required.

Depending on the outcome of the initial assessment, you may be required to provide further advice to IPENZ, including advice on any systemic issues identified.

Documents enclosed
Please find a USB drive with the following documents enclosed with this letter:

1. Masterton District Council Property files (obtained from Masterton District Council) for:

    a. 57-65 Dixon Street [redacted]
    b. 96-120 Queen Street [redacted]
    c. 408 Queen Street [redacted]
    d. 57-56 Dixon Street [redacted]
    e. 57-65 Dixon Street [redacted]
    f. [redacted]

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2. A chronology of the building design and consent process for each of the above buildings.

3. IPENZ's Guidelines for Independent Advisors.

IPENZ has requested responses from the primary engineers involved in the development of these buildings (in particular, the engineers who signed the PS1 and PS4 producer statements). In addition, we have requested full copies of the files held by KOA Ltd to ensure that we have a complete copy of the designs and other relevant information. Once we have received this information this will also be forwarded to you to consider as part of your assessment.

If you find there is information missing, please contact me. We would prefer to try to provide all information before you complete your report.

Expert advice required
Please review the documentation provided and advise whether you consider the engineering designs were produced in accordance with accepted standards and consistent with what a reasonable engineer, in the same circumstances, would likely have done. Please provide your advice in relation to each building separately.

In each case, where relevant, please include comment on what standards applied and whether those standards were complied with.

If you consider that there has been a departure from accepted standards, please advise how significant a departure you consider it is and how it would be viewed by your peers.

In a situation where the designs were developed by a junior engineer and then the producer statement — design — PS1 signed by a CPEng please comment on what responsibility the CPEng had for reviewing the design and ensuring it was consistent with accepted standards.

If you note a conflict in the evidence, please provide your advice in the alternative. For example: whether the care was appropriate if scenario (a) was correct, and whether it was appropriate if scenario (b) was correct.

Please review IPENZ's Guidelines for Independent Advisors before providing your advice.

Conflict of interest
I understand that you do not have a personal or professional conflict in this case. However, if you become aware of a conflict, please disclose it and, where appropriate, give recommendations on another advisor.

Disclosure of advice
It is IPENZ's intention to provide a copy of your advice to the relevant parties involved for comment. In the event that this matter(s) proceeds to a disciplinary committee or appeal body your advice may be relied on and disclosed to relevant parties.

You may be asked to give oral evidence if a formal disciplinary hearing is held. However, only a small number of cases proceed to a formal hearing.

Please note that, in the event that a disciplinary action is upheld, it is IPENZ's practice to publish its reports and you will be named.

It is important that you do not enter into any discussions about your advice, in the unlikely event that the engineer complained about, or his or her lawyer or insurer, or the media, contacts you to challenge or debate your advice. If you are contacted please direct them to me.

Confidentiality
As part of your role in providing independent advice to IPENZ, you will receive confidential and private information about the complaint you are providing advice on. This includes personal information, and could also include intellectual property and private/confidential information relating to IPENZ projects or programmes. Certain rights attach to that kind of information, and it is important that these rights are protected. I **enclose** a Confidentiality Agreement for you to sign and return to me, agreeing to abide by these requirements.

Timeframe
I would be grateful if you would forward a signed copy of your advice, as well as an electronic copy by email. Please ensure that all the documents sent with this request are returned with your report. I look forward to receiving your report and an account for your services, as discussed, by **10 April 2017**.

Please do not hesitate to contact me by email: [redacted] or telephone [redacted] if you need any further information.

Yours sincerely

[redacted]
Investigations Lead

Enc: Terms of Reference
USB drive with property files from Masterton District Council
Chronology of design and construction for each building
Guidelines for expert advisors
Confidentiality Agreement

APPENDIX B

PRELIMINARY FINDINGS ON THE SIX MASTERTON BUILDINGS

Appendix B1

408 Queen St Building Preliminary Compliance Report

Site Address	408 (390) Queen Street, Masteron
Designer of Record	KOA / CP1 Author : K O'Connor (2/2003)
Revision	Draft #1 for IPENZ review dated 23/5/17
Prepared by	B J Brown CPEng

1.0 REFERENCE INFORMATION

1.1	Address	408 Queen Street, Masterton
1.2	Site Description	Lots 2 and 3, DP 74295
1.3	Architects	Proarch Architects ref 74295
1.4	Engineers	Kevin O'Connor & Associates (KOA) ref 102610
1.5	PS1/PS4 Author	KOA / Kevin O'Connor CPEng
1.6	Building Consent	030125 dated 12/3/03
1.7	Third Party Reviewer (if any)	None
1.8	Code Compliance Certificate	4/9/03

2.0 BACKGROUND

2.1 Proarch drawings reference 2761/A Series relate to this project, with drawings A2 (Plan), A3 (Elevations), and A4 (Sections AA, BB, CC) describing the constructed form of the building.

2.2 KOA drawings reference 102610/S Series describe the structural content.

2.3 Other KOA drawings are included on the Council file to describe the site development and associated service works etc.

2.4 Geotechnical report information provided with design - none.

3.0 BUILDING DESCRIPTON

3.1 Single level commercial/industrial building, rectangular in plan, constructed in concrete (including precast tilt-up) and structural steel.

Plan dimensions : Length (L) 26.8 m (grids 1-4)
Width (B) 21.7 m (grids A-B)
Height (H) varies 6.5 m to 5.6 m

3.2 North (grid A) boundary wall formed in precast concrete panels connected to steel PFC transom beam, with south wall framing being lightweight metal cladding supported off steel girts.

3.3 Steel UB portal frames on grids 2, 3, and 4 span across the building (grids 1-2) with transom beams supporting the precast panels.

3.4 Foundations are 300 mm thick square pads up to 1 m square support portal frame columns, and the precast wall panels on external grids A, 4, and B.

3.5 Floor is 150 mm reinforced ground bearing concrete slab, incorporating an edge thickening to facilitate the wall panel/foundation junction.

3.6 Wall panel to foundation connection is made using concrete TCM inserts cast into the base of the precast panels, viz :

(a) boundary wall : TCM12 inserts at 150 c/c – two rows; and

(b) other walls : TCM12 inserts at 350 c/c – two rows.

4.0 KOA BUILDING DRAWINGS

102610/S1	Foundation/Floor Slab Plan
102610/S2	Foundation Details
102610/S3	Roof and Canopy Plan
102610/S4	Wall Elevation Grid 1 and Section Grid 2
102610/S5	Section Grid 3 and Wall Elevation Grid 4
102610/S6	Wall Elevations Grid A and B
102610/S7	Steel Details
102610/S8	Panel Diagrams (Concrete Walls)
102610/S8A	External Secure Yard Panels (Concrete Walls)

5.0 COMPLIANCE SUFFICIENCY CHECKS

5.1 Compliance requires justification using the following verification methods.

(a) Loadings NZS 4203:1992

(b) Concrete NZS 3101:1995

(c) Steel NZS 3404:1997

5.2 Structural components potentially undercapacity are tabulated for each of the transverse and longitudinal framing directions in Table 49158/1 "Lateral Force Resisting System Analysis" below.

Table 49158/1 : Lateral Force Resisting System Analysis

Transverse Direction	Resistance By	Compliance Sufficiency/Remarks
T(1) Portal frames 2, 3	Frame action	Grid A Type B panel outer column flange stabilised only by 12 mm flange plates into 2No TCM 16 reference section 14/54 and Type B panel on drawing S8 Inner flange to UB rafter adjoining portal knee, eg grid B, subject to 30% moment redistribution requires tie-backs to purlins NB : Reference KOA calculations p9
T(2) Foundation connections precast wall panels Grid A	Flexural restraint for vertical cantilever from foundation	No top propping restraint provided by transom beam to panel, leaving boundary eave detail 13/A5 vulnerable to damage (SLS)
T(3) Precast wall panels grids 1, 4	Shear wall - in-plane shear	Foundation connection detail (TCM 12 at 150 c/c) essentially brittle and requires verification for concurrent face load flexure and in-plane shear effects
T(4) Precast doorway lintel/grid 1	Type C panel supported by small site-welded weld plates (WP3)	Site-welded WP3 plates are susceptible to overload from shrinkage/thermal strains, plus seismic face load effects NB : Fail critical detail which is potential CSW

Transverse Direction	Resistance By	Compliance Sufficiency/Remarks
Longitudinal Direction	**Resistance By**	**Compliance Sufficiency/Remarks**
(L1) grid B/2-3 (125 SHS 8) inclined strut	Single strut/tie relies on tension/compression action which is vulnerable in seismic overload case	Compression only brace - eccentric cleat connection 15/S6 and 22/S6 potentially unstable under overload and will require evaluation assuming $\mu = 1$, $S_p = 1$ response NB : Tributary loads from top precast panels from grid 1 uncertain
(L2) grid A/1-4 interconnected shear panels	In-plane shear	Panel/panel weld plate connections vulnerable to overload from shrinkage/thermal overload effects

5.3 As part of my review, I have prepared preliminary calculations for the TCM inserts forming the wall/foundation junction for resistance of both out-of-plane (face/bending) loads and in-plane (shear) loads in a typical panel.

5.4 These calculations suggest that these TCM connections are marginally under-designed - ie say 80% of a fully compliant requirement - refer Appendix C5 calculations ref 49158/1/01-05.

6.0 SUMMARY / CONCLUSION

6.1 Preliminary evaluation suggests that the following structural components may be under capacity.

(a) Boundary wall grid A', 1 and 2 - base restraint to vertically cantilevered precast concrete wall panels.

Query TCM capacity insufficient to resist flexural overload in vertically spanning wall panel under face loading, particularly when combined with reaction to in-plane shear.

(b) End connection to inclined strut grid B/2-3 is vulnerable to instability when subject to severe seismic design actions.

(c) Interconnected shear panels on grids 1 and A rely on site-welded embedments which are vulnerable to overload from temperature/shrinkage strains, with lintel panel (grid 1/A-B) being particularly vulnerable.

NB : This is potential hazard to egress from building under seismic load case.

6.2 Further review of these details is recommended.

Appendix B3

57-65 Dixon St Preliminary Compliance Report

Site Address	Cnr Dixon & Church Streets, Masteron
Designer of Record	KOA / CP1 Author : K O'Connor (10/2006)
Revision	Draft #1 for IPENZ review dated 23/5/17
Prepared by	B J Brown CPEng

1.0 REFERENCE INFORMATION

1.1	Address	cnr Dixon & Church Streets, Masterton
1.2	Site Description	Lot 13, DP 10491
1.3	Architects	Proarch Architects ref 3986
1.4	Engineers	Kevin O'Connor & Associates (KOA) ref 110286
1.5	PS1/PS4 Author	KOA / Kevin O'Connor dated 6/10/06
		[redacted] [redacted]
1.6	Building Consent	070701 dated 11/9/07
1.7	Third Party Reviewer (if any)	Spencer Holmes / IG (11/06)
1.8	Code Compliance Certificate	Issued 3/4/07

2.0 BACKGROUND

2.1 Proarch drawings reference 3986/A Series relate to the project, with drawings A201 (Plan), A300 (Elevations), and A402 (Sections) describing the constructed form of the building.

2.2 KOA drawings reference 110286/S Series describe the structural content, as per the drawing list following.

2.3 Other KOA etc drawings are included on the Council file to describe the site development and associated utility service works etc.

2.4 Geotechnical report information provided - none.

3.0 KOA DRAWINGS

110286/S01(A)(C)	Foundation Plan
110286/S02(A)(D)	Slab Plan
110286/S03(A)(B)	Mezzanine Plan
110286/S04(A)(-)	Roof Plan
110286/S05(A)(B)	Canopys
110286/S06(A)(E)	Elevation Grid 1, 2
110286/S07(A)(C)	Elevation Grid 3, 4
110286/S08(A)(-)	Elevation Grid 5, 6
110286/S09(A)(-)	Elevation Grid 7, A
110286/S10(A)(B)	Elevation Grid B, C, D
110286/S11(A)(-)	Foundation Details 1
110286/S12(A)(-)	Foundation Details 2
110286/S13(A)(B)	Steel Details 1
110286/S14(A)(D)	Steel Details 2
110286/S15(A)(C)	Steel Details 3
110286/S16(A)(-)	Aircon Platform Unit

Drawings list also includes Standard Details S01(A) and S02(A) for concrete and masonry.

NB : Building consent issue corresponds to rev A (2/10/2010). Subsequent issues incorporate revisions as noted (up to 12/2010).

4.0 BUILDING DESCRIPTION

4.1 Rectangular building orientated NE-SW with re-entrant floor plan, connecting to an existing (smaller) building along the southwest side. The new building comprises two areas -

(a) Northeast (grids A-B/4-7) 14.0 m L × 18.4 m W × 6.0 m H

(b) Southwest (grids A-D/1-4) 28.0 m L × 21.3 m W × 6.3 m H

- and includes mezzanine over part area on SE side - dimensions 21.3 m L × 5.8 m W × 2.7 m H.

4.2 External walls comprise 150 mm thick precast panels along SW, NE, and part NW elevation (these being supported at regular intervals on bored rc piles to shallow depth) with lightweight metal cladding to the other elevations.

4.3 All portal frames plus mezzanine floor are similarly pile supported (although one lightweight clad wall is on strip footing).

4.4 Roofing is profiled metal supported on CR steel purlins, and the roof diaphragm comprises cross-braced R16 rods.

4.5 Southwest boundary to the new building abuts precast panel end walls of an adjacent building area, which provides lateral restraint to the main portals in the new building.

5.0 LATERAL FORCE RESISTING SYSTEM

5.1 The building incorporates the following lateral force resisting system.

(a) Transverse

(i) Steel UB moment frames on grids 2, 3, 4, 5, 6, and 7
(ii) Existing building wall adjacent, viz grid 1
(iii) 150 mm return wall panels 1/C-D and 2/C-D

(b) Longitudinal

(i) 150 mm precast concrete wall panels on grid A/1-7
(ii) R16 rod cross-braced to walls on grid B/4-5
(iii) 150 mm precast concrete wall panels on grid D/1-2
(iv) R16 rod cross-bracing to roof on panels A-B/4-5
(v) R16 rod cross-bracing to roof on panels B-C/2-3

(c) 150 mm precast wall panels stabilised against face loads by :

(i) 250×90 PFC transom at eaves level grid A/1-7
(ii) 250×90 PFC transom at eaves level grid D/1-2 (stabilised by return walls 1/C-D and 2/C-D)

57-65 Dixon St

5.2 Overall, the building layout and lateral force resisting system adopted is very similar to that for the Carpet Court building ref 49158/4 discussed separately in Appendix B4.

6.0 COMPLIANCE SUFFICIENCY CHECK

6.1 Compliance requires justification using the following verification methods.

(a) Loadings : NZS 4203:1992

(b) Materials

(i) Concrete : NZS 3101:1995
(ii) Steel : NZS 3404:1997

6.2 150 mm precast wall panel on gridline A (boundary)

(a) The 150 mm thick precast wall panel on this gridline spans vertically, and requires to be stable as a vertical cantilever under fire burnout load conditions.

(b) Because of this, the strength/robustness of the panel to foundation connection is critical, and because it incorporates potentially brittle cast-in reinforcing connection inserts, these must satisfy general design requirements for the type of connection which are discussed separately.

(c) Our review of KOA calculations for this junction suggests that :

(i) a force couple separated by 300 mm is being relied upon to resist overturning cantilever moments in the wall panel;
(ii) a significant restraining force needs to be generated along the bottom edge of the panel to balance the tension forces in the TCMs if this mechanism is to operate; however
(iii) the bottom of the panel is a "free edge" and there is no structural element (such as a continuous foundation beam with a rebated top edge to provide this reactantforce).

(d) As designed, this connection appears to provide :

(i) only some 30% of the moment demand required at foundation level; and
(ii) insufficient strength to reliably resist likely seismic face load demand, given that the top propping restraint will likely be of insufficient stiffness.

(e) If the 400 mm deep slab edge beam shown on Section 1/S01 could be shown to extend over the full span between the supporting piles, then the available lever arm acting with the TCM in the upper slab might well be able to reliably provide the necessary panel base moment restraint. However, Details 5/S01 and 6/S01 suggest that it was not intended to be constructed this way, and hence the under-capacity I have referred to is likely to be present.

6.3 Canopy Cantilever Rafter Connection to Column

This connection detail would appear to be under-designed as the tee stem loads the unstiffened face of the supporting 125×75×5 RHS post - refer Section 21/S13.

6.4 Transom/Panel Connection Drawing S15 - Detail 11/S09 (plus Section 12)

This connection has no tolerance in the transom/panel post-drilled bolting to give provision for in-plane deformation of the panels under in-plane seismic loading.

NB : This will be particularly important for the transom junction between tall and narrow Type P1 panels which will be likely to sustain large in-plane moments under seismic loading.

7.0 SUMMARY / CONCLUSION

7.1 My preliminary evaluation suggests that the following structural components within this building may be under capacity.

(a) Boundary wall on gridline A - base restraint to vertically cantilevered precast concrete wall panels

Query TCM capacity insufficient to resist flexural overstrength in vertically spanning panels under face loading - a situation made worse when combined with reaction to seismic in-plane shear load effects.

(b) Canopy rafter connection to column

Query sufficiency of unstiffened RHS face to withstand compression loads from tee stem of cantilever rafter supported by it.

(c) Transom/wall panel connection

Inadequate provision for movement between panel and transom when panel undergoes in-plane deflection under seismic loads.

7.2 Confirmation (or otherwise) of these items will require a further analysis and assessment.

Appendix B4

57-65 (Lot 9) Dixon St Preliminary Compliance Report

Site Address	57-65 Dixon Street (cnr Dixon & Church Streets), Masteron
Designer of Record	KOA / CP1 Author : K O'Connor (10/2010)
Revision	Draft #1 for IPENZ review dated 23/5/17
Prepared by	B J Brown CPEng

1.0 REFERENCE INFORMATION

1.1	Address	57-65 Dixon Street (cnr Dixon & Church Streets), Masterton
1.2	Site Description	Pt Lot 8, DP 10491
1.3	Architects	Proarch Architects ref 3986
1.4	Engineers	Kevin O'Connor & Associates (KOA) ref 110286
1.5	PS1/PS4 Author	Unknown dated 28/10/10 ([redacted])
1.6	Building Consent	100547 dated 30/10/10
1.7	Third Party Reviewer (if any)	Spencer Holmes / Carl Ashby
1.8	Code Compliance Certificate	Issued 7/11/11

2.0 BACKGROUND

2.1 Proarch drawings reference 3986/A Series relate to the project, with drawings A201-6 (Plans), A300 (Elevations), and A400 (Sections) describing the constructed form of the building.

2.2 KOA drawings reference 110286/S Series describe the structural content, as per the drawing list following.

3.0 KOA DRAWINGS

110286/S01	Foundation Plan
110286/S02	Slab Plan
110286/S03	Mezzanine Plan
110286/S04	Roof Plan
110286/S05	Canopys
110286/S06	Elevation Grid 1, 2
110286/S07	Elevation Grid 3, 4
110286/S08	Elevation Grid 5, 6
110286/S09	Elevation Grid 7, A
110286/S10	Elevation Grid B, C, D
110286/S12	Foundation Details 2
110286/S13	Steel Details 1
110286/S14	Steel Details 2
110286/S15	Steel Details 3
110286/S16	Aircon Platform Unit
110286/SD1	Standard Details Concrete and Masonry
110286/SD2	Standard Details Structural Steel

4.0 BUILDING DESCRIPTION

4.1 Rectangular building orientated NE-SW with re-entrant floor plan, connected to an existing (smaller) building along the southwest side. The new building comprises two areas -

(a) Northeast (grids A-B/4-7) 14.0 m L × 18.4 m W × 6.0 m H

(b) Southwest (grids A-D/1-4) 28.0 m L × 21.3 m W × 6.3 m H

- and includes mezzanine over part area on SE side - dimensions 21.3 m L × 5.8 m W × 2.7 m H.

4.2 External walls comprise 150 mm thick precast panels along SW, NE, and part NW elevation (these being supported at regular intervals on bored rc piles to shallow depth) with lightweight metal cladding to the other elevations.

4.3 All portal frames plus mezzanine floor are similarly pile supported (although one lightweight clad wall is on strip footing).

4.4 Roofing is profiled metal supported on CR steel purlins, and the roof diaphragm comprises cross-braced R16 rods.

4.5 Southwest boundary to the new building abuts precast panel end walls of an adjacent building area, which provides lateral restraint to the main portals in the new building.

5.0 LATERAL FORCE RESISTING SYSTEM

5.1 The building incorporates the following lateral force resisting system.

(a) Transverse

(i) Steel UB moment frames on grids 2, 3, 4, 5, 6, and 7
(ii) Existing building wall adjacent, viz grid 1
(iii) 150 mm return wall panels 1/C-D and 2/C-D

(b) Longitudinal

(i) 150 mm precast concrete wall panels on grid A/1-7
(ii) R16 rod cross-braced to walls on grid B/4-5
(iii) 150 mm precast concrete wall panels on grid D/1-2
(iv) R16 rod cross-bracing to roof on panels A-B/4-5
(v) R16 rod cross-bracing to roof on panels B-C/2-3

(c) 150 mm precast wall panels stabilised against face loads by :

(i) 250×90 PFC transom at eaves level grid A/1-7
(ii) 250×90 PFC transom at eaves level grid D/1-2 (stabilised by return walls 1/C-D and 2/C-D)

5.2 Overall, the building layout and lateral force resisting system adopted is very similar to that for the Beaurepairs building ref 49158/3 discussed separately in Appendix B3.

6.0 COMPLIANCE SUFFICIENCY CHECK

6.1 Compliance requires justification using the following verification methods.

(a) Loadings : AS/NZS 1170 Set (including NZS 1170.5)

(b) Materials

(i) Concrete : NZS 3101:2006

(ii) Steel : NZS 3404:1997

6.2 150 mm precast wall panel on gridline A (boundary)

(a) The 150 mm thick precast wall panel on this gridline spans vertically, and requires to be stable as a vertical cantilever under fire burnout load conditions.

(b) Because of this, the strength/robustness of the panel to foundation connection is critical, and because it incorporates potentially brittle cast-in reinforcing connection inserts. These must satisfy general design requirements for the type of connection which are discussed separately.

(c) Our review of KOA calculations for this junction suggests that :

(i) a force couple separated by 300 mm is being relied upon to resist overturning cantilever moments in the wall panel;
(ii) a significant restraining force needs to be generated along the bottom edge of the panel to balance the tension forces in the TCMs if this mechanism is to operate; and
(iii) the bottom of the panel is a "free edge", and there is no structural element (such as a continuous foundation beam with a rebated top edge) to provide this reactant force.

(d) As designed, this connection appears to provide :

(i) only some 30% of the moment demand required at foundation level; and
(ii) insufficient strength to reliably resist likely seismic face load demand, given that the top propping restraint from the transom will be relatively flexible.

(e) If the 400 mm deep slab edge beam shown on Section 1/S01 could be shown to extend over the full span between the supporting piles, then the available lever arm acting with the TCM in the upper slab might well be able to reliably provide the necessary panel base moment restraint. However, Details 5/S01 and 6/S01 confirm that it was not intended to be constructed this way, and hence the undercapacity I refer to is likely to be present.

6.3 Canopy Cantilever Rafter Connection to Column

This connection detail would appear to be under-designed as the tee stem loads the unstiffened face of the supporting 125×75×5 RHS post - refer Section 21/S13.

6.4 Transom/Panel Connection Drawing S15 - Detail 11/S09 (plus Section 12)

This connection has no tolerance in the post-drilled bolting to the transom/panel connection to permit in-plane deformation of the panels under in-plane seismic loading.

NB : This will be particularly important for the transom junction between tall and narrow Type P1 panels which will be likely to sustain large in-plane moments under seismic loading.

6.5 This connection is moment resisting and will not allow free rotation at the UB beam end. The bolt within the end plate are designed for shear only, without acknowledging the concurrent tension forces that will be generated by rotation support rotation and resulting flexure of the end plate.

In my opinion, the bolts to this connection as detailed are under-strength, and a web-side plate connection which allows end rotation should be substituted for it.

Refer KOA drawings S03/Grid A' and A'' plus KOA/EL calculations page L3.

6.6 As part of my review, I have prepared preliminary calculations for the TCM/RB inserts forming the wall/foundation junction for resistance of both out-of-plane (face/bending) loads and in-plane (shear) loads in a typical panel.

6.7 These calculations suggest that these TCM/RB connectors are significantly under-designed in terms of resistance to rupture under severe seismic (ULS) loads - ie say 20% of a fully compliant requirement - although collapse will be avoided in the general case due to the presence of transom beams above.

NB : Refer Appendix C5 calculations reference 49158/4/01-05.

7.0 SUMMARY / CONCLUSION

7.1 My preliminary evaluation suggests that the following structural components within this building may be under capacity.

(a) Boundary wall on gridline A - base restraint to vertically cantilevered precast concrete wall panels

Query TCM capacity insufficient to resist flexural overstrength in vertically spanning panels under face loading - a situation made worse when combined with reaction to seismic in-plane shear load effects.

(b) Canopy rafter connection to column

Query sufficiency of unstiffened RHS face to withstand compression loads from tee stem of cantilever rafter supported by it.

(c) Transom/wall panel connection

Inadequate provision for movement between panel and transom when panel undergoes in-plane deflection under seismic loads.

7.2 Confirmation (or otherwise) of these items will require a further assessment.

Appendix B5

196-120 Queen St Preliminary Compliance Report

Site Address	196-120 Queen St, Masteron
Designer of Record	KOA / CP1 Author : K O'Connor (6/2011)
Revision	Draft #1 for IPENZ review dated 23/5/17
Prepared by	B J Brown CPEng

1.0 REFERENCE INFORMATION

1.1	Address	196-120 Queen St, Masterton
1.2	Site Description	Lot 1, DP 441493
1.3	Architects	Proarch Architects ref 4047
1.4	Engineers	Kevin O'Connor & Associates (KOA) ref 111012
1.5	PS1/PS4 Author	KOA / Kevin O'Connor dated 21/6/2011
1.6	Building Consent	110268 dated 26/7/11
1.7	Third Party Reviewer (if any)	Spencer Holmes / PCS (7/2011)
1.8	Code Compliance Certificate	Issued 4/11/2013

2.0 BUILDING DESCRIPTION

2.1 Two-level commercial/retail building, rectangular in plan, in concrete, structural steel

Plan dimensions : Length (L) 36.6 m grids A-E; Width (B)8.15 m grids 1-2; Height (H) 7.1 m

2.2 NE boundary wall formed in precast concrete panels connected to steel PFC transom beam, with SW wall framing being lightweight metal cladding supported off steel girts.

2.3 Steel UB portal frames on grids A-E spaced at 8.6 m centres span across the building (grids 1-2). These act integrally with precast wall panel acting as pseudo-columns supporting UB rafters on grid 2.

2.4 First floor is structural steel UB supporting timber-framed floor.

2.5 Floor is 150 mm reinforced concrete slab at ground level, with regularly spaced piled foundations to support the precast concrete walls and steel UB columns. Foundation 750 mm diameter bored are piles to shallow depth, viz 2.0 m.

3.0 BUILDING DRAWINGS

3.1 Proarch Architects ref 4047/A Series showing plans, elevations, sections, and details.

3.2 Structural engineers drawings ex KOA reference 111012/S Series dated 6/2011

S1(A) Foundation / Slab Plan
S2(A) Mezzanine Floor Plan and Section
S5(A) Roof Plan/Canopy Plan
S10(A) Elevation on Grid A
S11(A) Elevation on Grid B
S12(A) Elevation on Grid C
S13(A) Elevation on Grid E
S14(A) Elevation on Grid E
S15(A) End Wall Elevation
S16(A) Elevation on Grid 1

S17(A) Elevation on Grid 2
S18(A) "Cube" Framing Elevations and Plans (Grid A/1-2)
S20(A) Details 1 - Foundation
S21(A) Details 2 - Foundation
S22(A) Details 2 - Steelwork
S23(A) Details 4 - Steelwork
S24(A) Details 5 - Steelwork (untitled)

NB : Designer identified by initials IG/EL on drawing title block.

4.0 STRUCTURAL CONCEPT

4.1 Compliance requires justification using the following verification methods.

(a) Loadings : AS/NZS 1170 Set, including NZS 1170.5 seismic actions standard

(b) Materials

(i) Concrete : NZS 3101:2006
(ii) Steel : NZS 3404:1997

4.2 Provision of regularly-spaced transverse steel UB portal frames to stabilise two levels of 150 mm precast concrete wall panel spanning vertically.

(a) Panels are supported at top by horizontal transom beam (PFC) spanning between the portals. 3No post-drilled bolt fixings connect each panel to the transom.

(b) Panels cantilever above foundation and are stabilised for the fire burnout condition by 1500 wide × 200 deep footing cast integrally within the floor slab.

(c) The precast panel to foundation slab connection is made using proprietary TCM-12 concrete inserts spaced at 200 c/c.

Representative transverse frames are shown on drawings S10 (Grid A) through S15 (End Wall) with the panel/foundation TCM connection detailed on drawing S20/section 1-S1 and drawing S21/section 1-S1.

NB : These sections are located on the floor plan drawing S1.

4.3 Stability in the longitudinal direction is provided on grids 1 or 2 by discrete 150 mm thick shear walls (of varying lengths either full height or up to first floor) as shown on drawings S16-17.

The panel foundation connection to these walls uses the same arrangement of concrete inserts, viz TCM-12/200 c/c, as is used for the Grid A/face loaded panels, although these are acting to resist in-plane longitudinal shear/overturning forces. In those areas where panels do not extend full height, longitudinal stability is provided by R24 rod bracing, eg grids 1/A-B.

4.4 Verticality spanning 150 mm thick precast panels are typically reinforced with :

(a) Y16-300 vertical reinforcing bar; and

(b) Y12-400 horizontal reinforcing

- with single Y16 bars to the full panel perimeter.

4.5 The 360UB45 portal rafter/150 mm precast panel connection detailed on drawing S24 Detail 2/S11 incorporates a cast in weld plate with a web site plate connection that would appear to be sufficiently flexible to limit the moment demand likely to be generated by the beam.

5.0 PRELIMINARY ASSESSMENT OF STRUCTURAL ELEMENTS REQUIRING FURTHER COMPLIANCE REVIEW

5.1 TCM Panel/Foundation Connections

150 mm precast panels/sufficiency of TCM connection to the 200 mm thick stabilising foundation are required to have :

(a) sufficient TCM connections to generate the required tension and/or shear resistance to withstand the design earthquake face load moment demand (including overstrength); and

(b) as for (a) but including concurrent shear panel overturning when uplift forces occur when associated with longitudinal earthquake.

5.2 Refer separate model calculations which indicate that the TCM panel connection detail shown on the drawings for this building will be subject to both tension and shear demand, particularly under 45° earthquake design cases, and are unlikely to exhibit the required capacity to satisfy the relevant design standards, eg NZS 3101:2006 section 17..

5.3 Drawing 16(A)/Elevation Grid 1 : The R24 rod wall bracing shown on this drawing is a major structural element, but does not show end connections details to describe how the rod connects into the anchoring floor/roof etc elements etc.

5.4 Fire Burnout Design Case

(a) For load cases involving fire burn out, which requires the designer to ignore the stabilisation effects from the non-FRR steel frame elements, the panels must exhibit adequate capacity to cantilever vertically up from the foundation.

Our review finds that :

(i) the panel to foundation M* and V* demand calculated for the fire burn out case is correctly assessed; but

(ii) the ØM overstrength resistance calculated by the designer is incorrectly calculated due to the assumption that a force couple separated by 300 mm exists when clearly it does not.

NB : Panel base moment resistance actually provided is around one-third that required to cover the demand for this design aspect.

(b) It may be that a separate fire engineering-based analysis might be able to verify the fire rating capacity of the non-FRR transverse portal frames, but, without this justification, the design is non-compliant in terms of protecting other property from the NW boundary (grid 1) wall collapsing outward under fire burn out load conditions.

6.0 SUMMARY / CONCLUSION

6.1 I have listed in item 5.0 above areas where I consider structural components within this building may be under capacity.

6.2 Confirmation (or otherwise) of these items will require further analysis and assessment.

196-120 Queen St
196-120 Queen St

Appendix B6

Cnr Dixon and Church St Preliminary Compliance Report

Site Address	57-61 Dixon Street (cnr Dixon & Church Streets), Masterton
Designer of Record	KOA / CP1 Author : K O'Connor (12/2013)
Revision	Draft #1 for IPENZ review dated 232/5/17
Prepared by	B J Brown CPEng

1.0 REFERENCE INFORMATION

1.1	Address	57-61 Dixon Street (cnr Dixon & Church Streets), Masterton
1.2	Site Description	Lots 1 and 2, DP 436307
1.3	Architects	Proarch Architects ref 4519/A Series
1.4	Engineers	Kevin O'Connor & Associates (KOA) ref 113352 (12/2013)
1.5	PS1/PS4 Author	(1) Kevin O'Connor (unknown) (2) Mr Y (1/4/14)
1.6	Building Consent	140105 dated 16/4/14
1.7	Third Party Reviewer (if any)	Beca 10/4/14
1.8	Code Compliance Certificate	11/9/2014

2.0 BUILDING DRAWINGS

2.1 Proarch drawings reference 4519/A Series relate to the project, with drawings A201-6 (Plans), A300 (Elevations), and A400 (Sections) describing the constructed form of the building.

2.2 KOA drawings reference 113352/S Series dated 12/2013 describe the structural content, as per the drawing list following.

NB : Designer identified on calculations/drawings by initials "CB".

- S00 Cover page / Structural Model Perspective / Drawing Register
- SD1 Standard Details / Concrete
- SD2 Standard Details / Structural Steel
- S01 Foundation / Slab Plan
- S02 Foundation / Slab Details 1
- S03 Foundation / Slab Details 2
- S04 Roof Plan
- S05 Elevations
- S06 Steel Details 1
- S07 Steel Details 2
- BR01 Bracing Plan

NB : Other KOA drawings in set, but these relate to siteworks, services, etc.

3.0 BUILDING DESCRIPTION

3.1 Single level commercial/office building who footprint consists of two overlapping rectangles, generally aligned N-S, with eastern side located in close proximity (within 2 m) to eastern property boundary.

3.2 Plan Dimensions

Length (grids 1-5/4)	17.85 m
Width (grids B-D/1-2)	7.3 m
Width (grids A-D/2-4)	12.15 m
Width (grids A-C/4-5)	8.2 m

3.3 Foundation conditions comprise gravels (with allowable bearing 300 kPa) which allows use of :

(a) 150 mm thick concrete ground slab at ground floor;

(b) foundations for perimeter walls comprising edge thickening; and

(c) large pad foundations supporting transverse rc cantilever shear walls.

3.4 External fabric comprises :

(a) metal profile roof - monoslope on CR steel purlins; and

(b) lightweight wall cladding on timber/steel framing (or precast concrete walls).

3.5 Lateral force resisting system comprises :

(a) between gridline B-D (shop area) - cantilevered precast panels;

(b) between gridline A-B - timber-framed shear walls; and

(c) light gauge metal cross-bracing to continuous roof.

4.0 STRUCTURAL CONCEPT

4.1 Gravity structure comprising various posts and beams supported off the ground-bearing slab.

4.2 Two distinct forms of superstructure across two halves of the building, comprising :

(a) N-E side (grids B-D/1-5) : 3No shear walls (running parallel on grids 1, 3, 5) of varying lengths supporting steel purlins spanning up to 9.3 m;

NB : Along with the steel purlins, major steel beams (one 410 UB over the showroom) act to connect the tops of the shear walls in the longitudinal direction.

(b) S-W side (grids A-B/1-5) light timber-framed walls and isolated columns supporting steel purlins.

4.3 Roof structure connects across both the above areas via a cross-braced diaphragm, formed from a series of overlapping light gauge steel cross-braces.

4.4 Lateral restraint is provided in the longitudinal (SW-NE) direction, and the transverse (NW-SE) direction -

(a) 3No shear walls supported on large spread footings which act as cantilevers under face loads or shear wall under in-plane loads for the NE half.

(b) Numerous plasterboard-lined timber-framed shear walls for the SW half.

4.5 Given the consenting date (2014), the operative design standards were :

(a) Loading - AS 1170 Set (including NZS 1170.5 Seismic Action); and

(b) Materials - Concrete Standards NZS 3101:2006, particularly section 12 (Walls).

5.0 COMPLIANCE SUFFICIENCY CHECK

5.1 Preliminary assessment highlights the significantly different lateral resistance concept operating in the two halves of the building, and the light and potentially flexible diaphragm at roof level which joints these two halves together.

5.2 Our preliminary review assessment has focussed on two areas.

5.2.1 150 mm thick cantilevered rc walls

(a) These walls are :

- Grid 1/B-D : Height 4.65 m AFL; Length 7.3 m
  Vertical reinforcing Y16 @ 150 c/c into 4700 Drossbach ducts 975 long
- Grid 3/C-D : Height 4.55 m AFL; Length 4.0 m
  Vertical reinforcing similar to walls 1 and/or 5
- Grid 5/B-C : Height 4.95 m AFL; Length 3.45 m
  Vertical reinforcing Y20 @ 175 c/c into 5000 Drossbach ducts 1200 long

(b) Foundation for each are 2200 to 2400 wide and 600 deep to provide base fixing, and the foundation/wall connection is via grouting into Drossbach ducts referred to above.

(c) Whilst the floor slab abuts this cantilevered walls at some 400 mm (minimum) above the top of the footing, there is no reliance on the floor slab to stabilise the foundation.

(d) Preliminary calculations suggest that :

(i) in-plane loading generates shear demand of around 0.15 MPa which is significantly less than the dependable shear resistance of the concrete (0.76 MPa);
(ii) face loading generates ULS deflection of the order of 100 mm at roof level, this being less than the upper bound (125 mm or so) permitted by the seismic standard; and
(iii) the supporting foundation (ie 2.2 m wide) remains stable and the ground contact presses 400 kPa are within the capacity of the supporting ground.

(e) Given the size of these wall panels and the relative low seismic demand on them, I consider them adequate within themselves, although there may well be issue with the effects of "top of wall" rotation under face loads (refer item 5.2.2 below).

5.2.2 Primary roof level beam 410 UB 54 (grid D/1-3)

(a) 410 UB spans 9.3 m between bolted end plate supports formed by post-drilled/epoxied bolt fixings to precast wall panels on grids 1 and 3.

(b) Beam end connections are essentially moment resisting, although bolts are only designed for shear. Anticipate beam end connections will be forced to rotate as wall panels deflect under face load, thereby inducing tension in bolts, potentially overloading the shear connection, with potential collapse failure consequences.

(c) Refer KOA calculations page E1 for end plate and page C20 for panel top deflection calculations.

(d) This connection should, in my view, incorporate a flexible end plate-type connection rather than a moment resisting one.

5.3 Precast Concrete Panel to Foundation Connection

(a) The primary cantilever walls in this complex incorporate stitching of vertical reinforcing bars through grouting within "Drossbach" ducts cast into the precast panel.

(b) Due to past quality assurance problems experienced across the industry with this type of detail, we recommend that the owner commission invasive tests to confirm the sufficiency of grouting within these ducts.

6.0 SUMMARY / CONCLUSION

6.1 Preliminary assessment highlights the significantly different lateral resistance concept operating in the two halves of the building, and the light and potentially flexible diaphragm at roof level which joints these two halves together.

6.2 There is a potential weakness in the beam to wall panel connection for the 9.3 m long roof beam (grid D/1-3) supporting the roof.

6.3 I recommend special consideration of KOA's verification of the sufficiency of the Drossbach duct grouting within the precast concrete cantilever panels on this project.

APPENDIX C

SUPPORTING INFORMATION

C1 Various industry references on eg Tilt Up Construction

R1 "*Guidelines for the Use of Precast Concrete in Buildings*" 1991
Report of a study group of the NZ Concrete Society and NZNSEE published by University of Canterbury, Christchurch, 174pp

R2 "*Review of Tilt-Up Construction Details*" JI Restrepo & R Park
NZ Concrete Society Conference, Auckland, October 1993 (p38-43)

R3 "*Tensile Capacity of Steel Connectors with Short Embedment Lengths in Concrete*" JI Restrepo-Posade, P Park
Research Report 93-6 Department of Civil Engineering, University of Canterbury, Christchurch, 1993 (51pp)

R4 "*Seismic Design Aspects for Tilt-Up Buildings*" JI Restrepo, FJ Crisafulli, R Park
SESOC Jnl Vol 9 No 2 Dec 1996 pp9-24

NB : Section (pp13-15) entitled "Tensile Strength of Wall to Suspended Slab Connections" etc and references the "ψ method" of predictions the expected cone pull out strength ref UC Research Report 96-11 (1996)

R5 "*Some Stability Issues for Tilt Up Precast Panels under In Plane Seismic Loadings*" BJ Brown
SESOC Journal Vol.10 No.1 (p34-42) June 1997

R6 "*Design of Slender Precast Wall Panels*" - BRANZ Study Report SR129 (2003)

R7 "*Determination of seismic design forces for slender precast slab stru*ctures" BJ Davidson
NZSEE Conference 2004 Paper No.14 (10pp)

R8 "*Review of Design and Construction of Slender Precast Concrete Walls*" RA Poole
Report to Department of Building & Housing, August 2005 (30pp)

R9 "*Design Guide* (2007) : *Slender Precast Concrete Panels with Low Axial Load*" GJ Beattie
BRANZ Limited, Wellington

R10 "*Design of Conventional Structural Systems Following the Canterbury Earthquakes*"
NZ Structural Engineering Society, SESOC Interim Design Standards : Practice Note (v4-12/2011)
(section 3 "*Concrete Walls*" p9-17 are applicable to this discussion)

R11 "*Slender Precast Wall Panels Interacting with Steel Portal Frames under Earthquake Loa*ds" Joo H Cho, Harrison Grierson Christchurch : Case Study Paper dated 2014 (source unknown)

R12 "*Seismic strengthening of commercial warehouse with slender precast concrete panels, utilizing knowledge from the observed performance of similar buildings during the 2010/2011 Christchurch earthquakes: A case study*" P Armaos and DM Thomson - NZSEE Conference 2016 (7pp)

C2 Requirements of relevant Loading Standards, eg Seismic Actions (NZS 4203:1992 Part 4, and NZS 1170.5 Parts 5 and 8)

1. SEISMIC EFFECTS (PART 4)

1.1 Site category - intermediate (b) s4.6.2.2

1.2 Derive seismic hazard coefficient $C_h(T,\mu)$ using Fig 4.6.1(b)

T = 0.45 sec s4.6.2.7(b)

C_h = 1.0 ($C_h\mu$ (0.45, 1.0) = 0.8

μ = 1.25 $C_{h\mu}$ (0.45, 1.25) = 0.69

μ = 3.0 $C_{h\mu}$ (0.45, 3.0) = 0.35

1.3 Derive $C_h(T_{1\mu})$ for various values of μ (range 1.0<μ<3.0) using Eqn 4.6.2

μ = C = $C_h(T_{1\mu})$SpRZLu Eqn 4.6.2

		S_p = 0.67	S_p = 1.0	
$C_h(T_{1\mu})$	= 0.80×0.67×1.0×1.2×1.0 =	0.643	0.959	for μ = 1.0
	= 0.69×0.67×1.0×1.2×1.0 =	0.554	-	for μ = 1.25
	= 0.35×0.67×1.0×1.2×1.0 =	0.281	-	for μ = 3.0

1.4 Parts ref 4.12

Brittle connections require dependable strength scaling by 1.5 ref cl 4.12.1.6 where panels failure put people at risk, eg egress ways

NB : Equivalent static method ref Eqn 4.8.1, 4.8.2

$F_1 = 0.92V \frac{Wihi}{EWihi}$ at Level 2, and $F_2 = 0.08V$ at $h_i = h_n$

1.5 Evaluation of scaling factor over global for parts

Separate evaluation using NZS 4203:1992 cl 4.12 derives the following scale factors for the design of major components with light industrial (LIB) type buildings.

(a) Mid-height of vertically spanning elements, eg precast concrete walls, with top (transom) support scale factor typically 0.8 on global coefficient.

(b) Elements located at roof level, eg roof diaphragm, scale factor typically 1× applied to global coefficient at roof level.

2. WIND EFFECTS (PART 5)

2.1 Site at Zone IV/Non-Directional (southern North Island)

$V = 46$ m/sec
$V_Z = VM_{LS}\, M_{ZCat}\, M_s M_t M_r$ TC = 3 H = 1.0 m
$= 46\times0.93\times0.83\times1.0\times1.0\times1.0$
$= 35.5$ m/sec
$Q_2 = 0.6\, V_Z^2 = 0.6\times35.5^2 = 0.756$ kPa Eqn 5.5.1

2.2 Pressure coefficients using :

Table 5.6.2(a)	Windward wall $C_{pc} = 0.7$		
	Leeward wall	$C_{pc} = -0.5$	h/d <0.5
	Side wall	$C_{pc} = 0.5$	
	Upwind roof (u)	$C_{pc} = 0.9$	
	Downwind (B)	$C_{pc} = -0.5$	

Table 5.6.4 Area reduction factor <low <1.0

NB : Ignore local pressure factor Ke for design of vertically spanning panels of moderate size, eg 3 m long.

Internal pressure coefficients - permeability = -0.30 - equal typically - refer Table 5.6.1

3. FIRE (cl 24.3.4(b))

Boundary wall stability - face load 0.5 kPa (ULS)

1. SEISMIC (NZS 1170.5:2004 Part 5 (Global structure as a whole and Part 8 (Parts))

(a) Site Category - shallow soil site - say Class C for T = 0.5 sec (Table 3.1) gives Ch (T) = 2.0

(b) Structure Period - assume T = 0.5 sec

(c) Seismic Hazard Factor - Z = 0.42 for Masterton, with (D = 6-10)

(d) Equivalent Static Method - ref cl 5.2

$C_d(T_i) = \frac{C(T_i)\, S_p}{k\mu}$ Eqn 5.2(1)

$S_p = 1.3 - 0.3\,\mu$ Eqn 4.4.2

using μ *- 1.0, 1.25, and 3.0* as expected range across all elements within the subject building

NB : Nominally ductile elements 1.0 μ <1.25
Limited ductile elements 1.25 μ <3.0
Brittle elements μ = 1.0

K_μ = $\frac{\mu - 1.0}{0.7}$ T, + 1.0 (T <0.7 sec)
R = 1.0

(e) Near fault considerations ref Table 3.1

N (T, D) = 1 + (N_{max}(T)) - [$\frac{20-D}{18}$] = 1.0 (R = 1.0)
Table 3.7 N_{max}(T) - [$\frac{1.0}{14}$]
N (T, D) = 1 + (1.0 + 1.0) - [$\frac{20-6}{18}$] (D = 6 km)
- so only affects longer period structures

(f) C_T = Ch(T) Z RN (T,D) - elastic site hazard spectrum
Ch(T) = 2.0 × 0.42 × 1.0 × 1.0 = 0.84 (Table 3.1 using spectral shape for Class C soil)
$C_d(T_i)$ = [$\frac{C\,(T_i) S_p}{k\mu}$], giving for expected range of μ, S_p values
= [$\frac{0.84 \times 1.0}{1.0}$] = 0.840 for μ = 1, S_p = 1
= [$\frac{0.84 \times 0.925}{1.1.78}$] = 0.659 for μ = 1.25, S_p = 0.925
= [$\frac{0.84 \times 0.70}{2.430}$] = 0.242 for μ = 3.0, S_p = 0.7

(g) Concurrently, because for nominally ductile and brittle structure, viz 1.0 μ <1.25, need to consider effects of combined X, Y action - ie 100% X, 30% Y, but cover for this by scale all/any seismic action by say 15% to cover off concurrency effects.

(h) Summary of seismic coefficients for structure as a whole

μ	$C_d(T_i)$	Scale 1.15, or 1.0*	Parameter used
1.0	0.840	0.97	$K\mu = 1.000$, $S_p = 1.0$, $T = 0.5$
1.25	0.659	0.76	$K\mu = 1.178$, $S_p = 0.925$, $T = 0.5$
3.0	0.240	0.24*	$K\mu = 2.43$, $S_p = 0.70$, $T = 0.5$

(i) Evaluation of scaling factor over global loads for "Parts"

Separate evaluation using NZS 1170.5 Part 8 derives the following scale factors to be used in the design for major components within light industrial (LIB) type buildings.

(i) Mid-height of vertically spanning elements, eg precast concrete wall panels, with top (transom) support scale factor typically 1.4 on global coefficient.
(ii) Elements located at roof level, eg roof diaphragm, scale factor typically 1× applied to global coefficient at roof level.

2. WIND EFFECTS (AS/NZS 1170.2, SECTION 2)

$V_{site}, \Delta = [V_R\, M_d\, [M_{Zcat}, M_S, M_E]$ (β = unidirectional)

where V_R derived from wind region A7 - Figure 3.1(B) using Equation 2.2

A7 : Unidirectional = 1.0
$V_{500} = 45$ m/s Table 3.4
$M_d = 1.0$, $M_{Zcat} = 0.83$ — site exposed
$M_S = 1.0$ N/A — TC3 from cl 4.2; assuming H <10 m
$M_t = 1.0$ (no topographic acceleration)

$V_{site} = 45$ m/sec × 1.0 × 1.0 × 0.83 = 37.4 m/sec

Dynamic pressure P = (0.5 × 1.2) [37.4 m/sec]^2 × C_{fig}
= 0.6 × (37.4)^2
= 0.837 kPa × C_{fig} as required

3. FIRE (cl *****)

Boundary wall stability - face load 0.5 kPa (ULS)

Document History

Prepared by :	FTL / B J Brown
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Communication status :	Confidential
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Fraser Thomas
ENGINEERS • RESOURCE MANAGERS • SURVEYORS

C3 Information on Subsoil Classification for the subject sites

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ENGINEERS • RESOURCE MANAGERS • SURVEYORS

MEMORANDUM / FILENOTE

Date: 21 April 2017 49158

To: Barry Brown

Subject: Site Subsoil Classification for corner of Dixon and Church Streets, Masterton

From: Alistair Stuart

As requested, we have undertaken a review to determine the site subsoil classification for the site in accordance with New Zealand Standards NZS 4203:1992 Code of Practice for General Structural Design and Design Loadings for Buildings and NZS 1170.5:2004 Structural Design Actions Part 5: Earthquake actions – New Zealand.

In carrying out the review of the site, reference has been made to the Institute of Geological and Nuclear Sciences map, scale 1:250,000, Geology of the Wairarapa Area, Geological Map 11.

The New Zealand Geological Map indicates that the site is underlain by moderately to well sorted alluvial flood plain gravel with minor sand and/or silt of the Quaternary Age. The geological map gives no indication as to the density or thickness of the alluvial deposits, however, our job based experience of similar deposits suggests that these deposits may be of 'medium dense' consistency and between 30 m to 40 m thick.

NEW ZEALAND STANDARD NZS 4203:1992

New Zealand Standard NZS 4203:1992 provides three site subsoil categories as outlined below:

4.6.2.2 *Site subsoil categories*

Site subsoil category (a) (Rock or very stiff soil sites)
Sites where the low amplitude natural period is less than 0.25 s, or sites with bedrock, including weathered rock, with unconfined compressive strength greater than or equal to 500 kPa, or with bedrock overlain by:

(i) Less than 20 m of very stiff cohesive material with undrained shear strength exceeding 100 kPa; or

(ii) Less than 20 m of very dense sand, with N_1 >30, where N_1 is the SPT (N) value corrected to an effective overburden pressure of 100 kPa; or

(iii) Less than 25 m of dense sandy gravel with N_1 >30.

Site subsoil category (b) (Intermediate soil sites)
Sites not described as category (a) or (c) may be taken as intermediate soil sites.

Site subsoil category (c) (Flexible or deep soil sites)
Sites where the low amplitude natural period exceeds 0.6 s, or sites with depths of soils exceeding the following values:

Soil type and description		Depth of soil (m)
Cohesive soil	Representative undrained shear strengths (kPa)	
Soft	12.5 – 25	20
Firm	25 – 50	25
Stiff	50 – 100	40
Very stiff	100 – 200	60
Cohesionless soil	Representative SPT (N) values	
Loose	4 – 10	40
Medium dense	10 – 30	45
Dense	30 – 50	55
Very dense	> 50	60
Gravels	> 30	100

In terms of the geotechnical definition in clause 4.6.2.2 of NZS 4203:1992, the site is unlikely to be site subsoil category (a) (Rock or very stiff soil site) as it is mapped as being underlain by alluvial materials anticipated to exceed the required 25 m thickness for dense sandy gravel.

Given the conclusions above, it is our opinion that the site should be classified as site subsoil category (b) (Intermediate soil sites) in terms of NZS 4203:1992 .

NZS 1170.5:2004, STRUCTURAL DESIGN ACTIONS, PART 5: EARTHQUAKE ACTIONS

NZS 1170.5:2004, Structural Design Actions, Part 5: Earthquake Actions (SNZ 2004) has been phased into practice between 2004 and 2008 and is the current standard. Seismic design actions for a site are defined using five site subsoil categories, as outlined below:

Class A – Strong rock

Class A is defined as strong to extremely-strong rock with:

(a) Unconfined compressive strength greater than 50 MPa; and
(b) An average shear-wave velocity over the top 30 m greater than 1,500 m/s; and
(c) Not underlain by materials having a compressive strength less than 18 MPa or a shear-wave velocity less than 600 m/s

Class B – Rock

Class B is defined as rock with:

(a) A compressive strength between 1 and 50 MPa; and
(b) An average shear-wave velocity over the top 30 m greater than 360 m/s; and
(c) Not underlain by materials having a compressive strength less than 0.8 MPa or a shear-wave velocity less than 300 m/s.

A surface layer of no more than 3 m depth of highly-weathered or completely-weathered rock or soil (a material with a compressive strength less than 1 MPa) may be present.

Class C – Shallow soil sites

Class C is defined as sites where:

(a) They are not class A , class B or class E sites; and
(b) The low amplitude natural period is less than or equal to 0.6 s; or
(c) Depths of soil do not exceed those listed in Table 3.2 (of NZS 1170.5 – shown below).

The low amplitude natural period may be estimated from four times the shear-wave travel time from the surface to rock, be estimated from Nakamura ratios or from recorded earthquake motions, or be evaluated in accordance with Clause 3.1.3.7 (of NZS 1170.5) for sites with layered subsoil, according to the hierarchy of methods given in Clause 3.1.3.1 (of NZS 1170.5).

Class D – Deep or soft soil sites

Class D is defined as sites:

(a) That are not class A , class B or class E sites; and
(b) Where low-amplitude natural period is greater than 0.6 s; or
(c) With depths of soils exceeding those listed in Table 3.2 (of NZS 1170.5 – shown below); or
(d) Underlain by less than 10 m of soils with an undrained shear-strength less than 12.5 kPa or soils with SPT N-values less than 6.

The low amplitude natural period may be determined in accordance with Clause 3.1.3.4 (of NZS 1170.5).

Class E – Very soft soil sites

Class E is defined as sites with:

(a) More than 10 m of very soft soils with undrained shear strength less than 12.5 kPa; or
(b) More than 10 m of soils with SPT N-values less than 6; or
(c) More than 10 m depth of soils with shear-wave velocities of 150 m/s or less; or
(d) More than 10 m combined depth of soils with properties as described in (a), (b) and (c) above.

TABLE 3.2 **[NZS 1170.5]**

MAXIMUM DEPTH LIMITS FOR SITE SUBSOIL CLASS C

Soil type and description		Maximum depth of soil (m)
Cohesive soil	**Representative undrained shear strengths (kPa)**	
Very soft	< 12.5	0
Soft	12.5 – 25	20
Firm	25 – 50	25
Stiff	50 – 100	40
Very stiff or hard	100 – 200	60
Cohesionless soil	**Representative SPT N values**	
Very loose	< 6	0
Loose dry	6 – 10	40
Medium dense	10 – 30	45
Dense	30 – 50	55
Very dense	> 50	60
Gravels	> 30	100

In terms of the geotechnical definitions in clauses 3.1.3.2 and 3.1.3.3 of NZS 1170.5, the site is not Class A (Strong rock) or B (Rock) as it is mapped as being underlain by alluvial materials estimated to have a compressive strength of less than 0.8 MPa (corresponding to an undrained shear strength less than 400 kPa).

Given the conclusions above, it is our opinion that the site should be classified as Class C (Shallow soil sites) in terms of NZS 1170.5 although a conservative classification of Class D (Deep or soft soil sites) in terms of NZS 1170.5 may be adopted given that no site specific geotechnical information was available at the time of this review.

Alistair Stuart

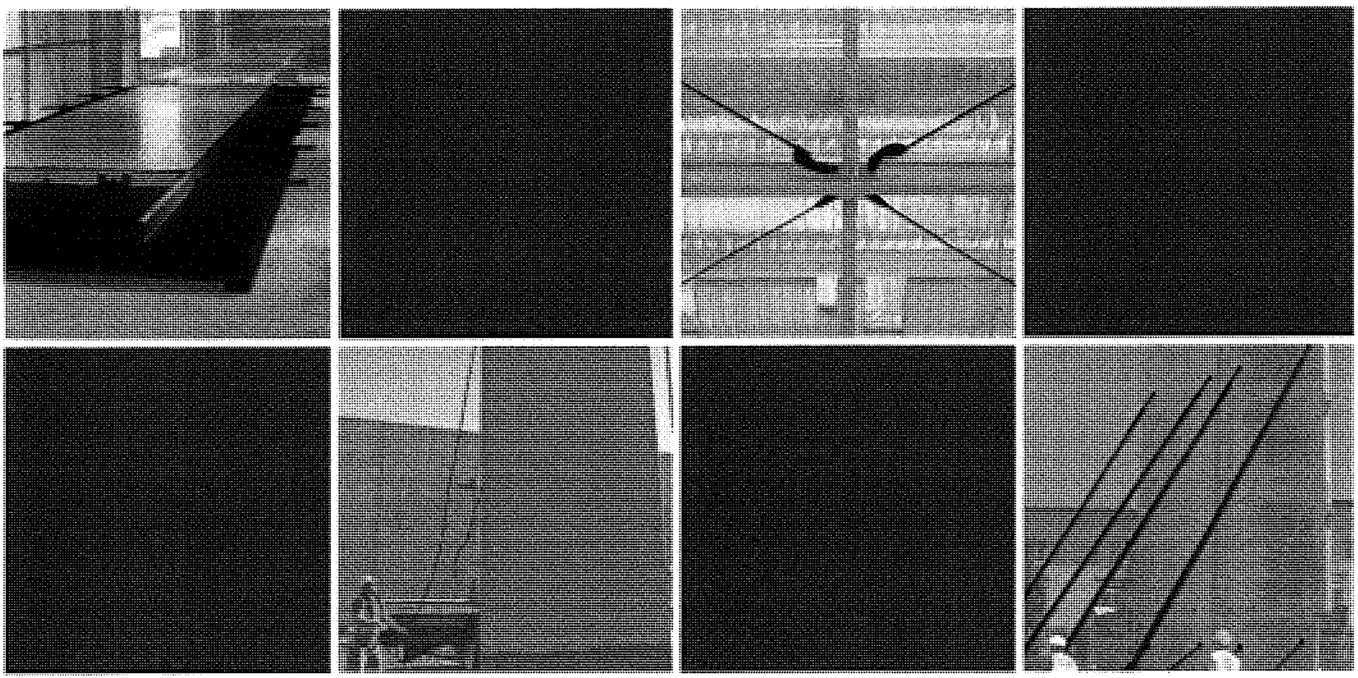
C4 Indicative Information on Proprietary Inserts for Reinforcement Anchorage in Precast Concrete Panels

reid

CONSTRUCTION SYSTEMS

Product Catalogue

& Design Guides



07/08

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![reid construction systems]

Reid™ Design Concepts for Reinforcement Anchorage

Over the past 10 years Reid™ Construction Systems have carried out research into the most accurate formula, based on embedment depth and concrete strength, for the pullout capacity of headed anchors in concrete. This research has resulted in the refinement of Reid's™ Threaded Inserts to ensure that in a minimum 30MPa concrete they are capable of breaking the reinforcing, ensure minimum slippage and have sufficient bearing area to prevent concrete crushing. The following table presents the information of Reid™ Insert Capacity based on the Haeussler formula and non cracked concrete.

Threaded Insert capacity in concrete

Minimum embedment depths for threaded inserts & footplate in 25Mpa and 30Mpa concrete

Table 31.

Product Code	Grade	L_1 Depth to develop Min Yield Strength (mm)		Char Min Yield Strength (kN)	L_2 Depth to develop Char Max Ult Strength (mm)		Char Max Ult Strength (kN)	Threaded Insert Length plus 8mm
		25Mpa	30Mpa		25Mpa	30Mpa		
RB12	500E	82	78	56.5	97	92	79	108
RBA16	500E	110	103	100.6	130	122	140.8	126
RB20	500E	137	129	157.0	162	153	219.9	156
RB25	500E	171	161	245.5	203	191	343.7	199
RB32	500E	219	206	402.0	260	244	562.9	-

Note 1: The adoption of embedment depth L^2 will ensure that the failure mechanism will be ductile rather than by brittle shear cone pullout. See note 3

Note 2: Embedment depth is calculated using the formulas developed by Haeussler.
The general form is given as $P = 0.972 \times L^2 \times B^{2/3}$

where: P = pullout capacity of shear cone in Newtons
L = effective embedment depth in mm
B = concrete compressive strength in MPa

(Test results have shown that pull out calculated with Haeussler will be about 15% conservative) (P.T. 2001)

Note 3: Screw in plastic nail plates recess the insert by 8mm

With modern design and construction practices, where thinner sections are used and anchorages are required in concrete tension zones that can be micro cracked, the above formula requires modification to represent the reduction in strength that will occur. While this is possible with the Haeussler approach, research in NZ has focused on an alternative formula for the prediction of cone pullout capacity which accommodates anchor centre, edge distances, material property variations and construction tolerances. Known as the ψ method, it is this design approach that is presented here to be consistent with NZ research.

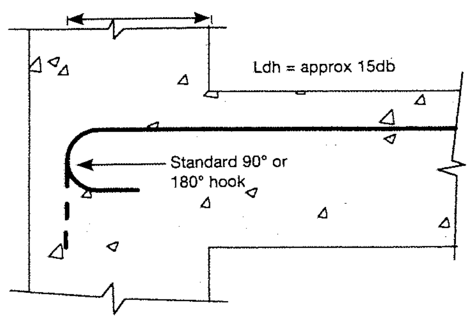
Why use a Reid™ Threaded Insert or Footplate when hooked and bent bars have always been used?

Is a common question asked by structural engineers when presented with the Reidbar™ system for the first time.

To answer this question some explanation is required:

Hooked bars and bent bars have been the standard method of providing anchorage for reinforcing steel in concrete construction where the standard bond length for a straight bar cannot be achieved. Concrete design codes account for this shorter *bond length* by specifying a minimum length, L_{dh} from the back of the bend, or hook, to the critical surface. The minimum length equation considers the effect of concrete strength, f'_c, and steel yield strength f_y.

Detail 36.



As designers and constructors become more familiar with the use of tilt-up and precast methods normal conservatism can be pushed to the limit. This is especially true with the current trend towards increasingly slimmer wall panels where the provision of an effective base anchorage for cantilever action is still required. Although bent starter bars are still widely used for this function it is not always possible to meet code requirements for minimum anchorage length in thin panels.

The Design Code NZS 3101 2006 and the previous code both draw attention to the issues related to concrete cone pullout of shallow embedment anchorage of hooked bars.

The minimum development length of 150 mm is removed from clause 7.3.14.2 and two new clauses added

"The development length, L_{dh}, calculated using eqn 7-11 shall apply when there is no likelihood of a failure mode of a pullout of a concrete cone from the volume of concrete in which the bar is anchored"

"If a cone of concrete pullout is likely then a rational analysis or suitable testing shall account for the proximity of the anchored bars to other loaded elements and to edges of elements"

Essentially these amendments say that one should not use hooked bars to develop full bar capacity **unless concrete cone pullout capacity exceeds the bar strength.**

reid™ CONSTRUCTION SYSTEMS

So how do I calculate concrete cone pullout ?

In 1993, NZ University of Canterbury research by *Restrepo-Posada and Park*[4] showed that the ψ-method can be used to predict the concrete cone capacity of hooked bar and headed stud type anchorages, provided that the correct embedment depth is defined. This design approach accounts for the influence of edge distance, bar spacing and micro cracking in tension zones, by applying reduction factors to the calculated concrete cone pull out capacity of the anchorage. To reduce the probability of premature brittle failures the approach also incorporates factors in the formula to account for likely variations in material strengths and construction tolerances.

The method and corresponding formula are set out in this manual in the form of a flow diagram on page 114 and is followed by a design example that compares the design of a "L" shaped hooked anchorage to that of a comparable sized Reid™ Threaded Insert anchor for a wall panel to foundation connection. (page 115.)

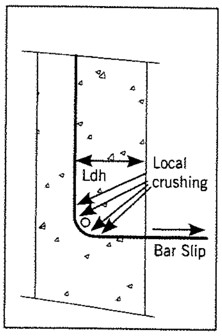
Importance of ductile failure

The importance of ductile failure should be appreciated, as it is essential to ensure that a brittle failure mechanism does not occur before a ductile failure, taking into account the possible material over strengths that can exist. In the design example it is shown that brittle failure of the anchorage will occur for both situations but in the case of the Threaded Insert it has enough capacity to ensure that the wall stem will have a ductile failure before cone pullout and thus provide a safe connection.

Anchorage slip

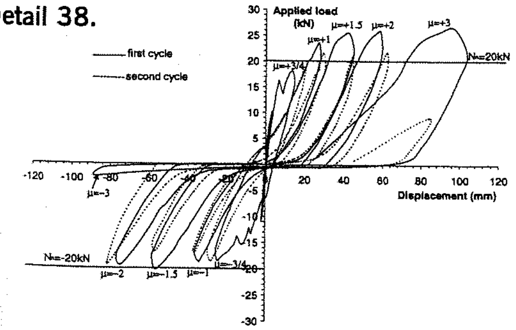
The ψ-method does not address slippage of the anchor. With hooked bars the inside of the hook causes local crushing of the concrete as the bar tries to straighten under load. Higher slippage of the reinforcing can occur compared to a headed anchor where the bearing stress under the head can be accommodated in the design of the product to minimize crushing.

Detail 37.



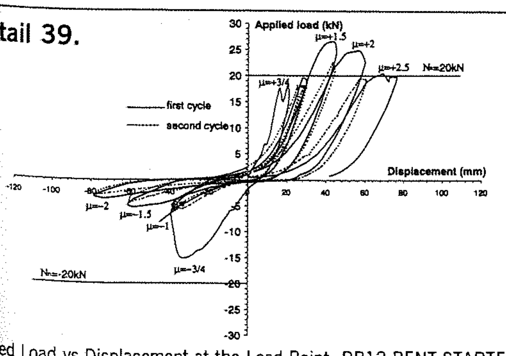
Research at NZ University of Auckland by *Maureen Ma in 1999 into Methods of Joining Precast Concrete components to form Structural Walls*[6] highlighted the performance of Reid™ Threaded Inserts compared to that of conventional hooked bar construction. The diagram below shows the test comparison between the two forms of anchorage in a wall panel to footing connection when subject to cyclic loading. It can be seen that the threaded inserts performed significantly better.

Detail 38.



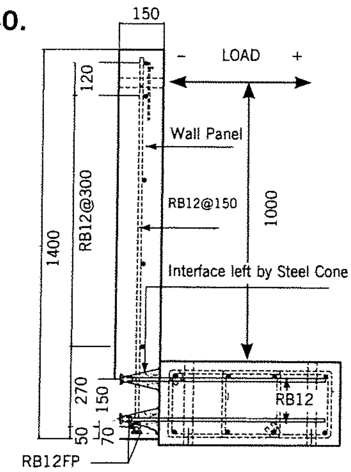
Applied Load vs Displacement at the Load Point RB12 FOOTPLATES

Detail 39.

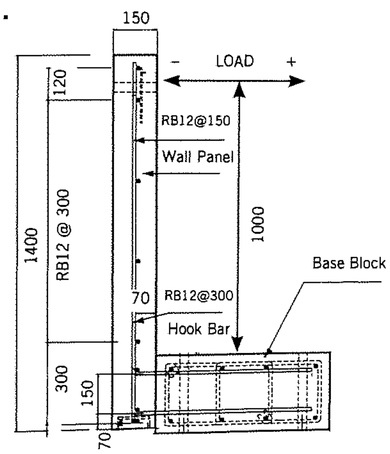


Applied Load vs Displacement at the Load Point RB12 BENT STARTERS

Detail 40.



Detail 41.



reid™ CONSTRUCTION SYSTEMS

Design Process for Cone Pullout

Can full development length Ldh and cover be achieved in the wall thickness
12mm dia – 214mm min thickness
16mm dia – 275mm min thickness
required for 30 Mpa concrete and
500 grade steel

yes → Conventional hooked bars possible

no → Use Reid™ Threaded Inserts
Min. wall thickness
RB12TI – 120mm
RBA16TI – 140mm

Do starter bars need site bending for access? — yes →

Do starter bars need bending for transport? — yes →

Use Reid™ Threaded Inserts

Still want to use hooked bar anchorages?

Calculate Concrete Cone Pullout from:

×0.23 FORMULA WRONG

$$h_e = \left(\frac{f_y}{\xi_R\sqrt{f_c'}}\right)^{\cancel{\frac{2}{3}}\ 4/3}\frac{d_b}{\cancel{4\eta}}$$ - to determine the effective embedment to yield the steel

or/ $$f_s = \xi_R\sqrt{f_c'}\left(\frac{h_e}{0.23\cancel{d_b}\ d_b^{4/3}}\right)^{1.5}$$ - to determine maximum steel stress without inducing cone pullout

where: f_c' = characteristic concrete strength – Mpa
f_y = characteristic steel yield strength - MPa
f_s = steel stress – Mpa
d_b = nominal steel diameter – mm
h_e = effective anchor embedment – mm

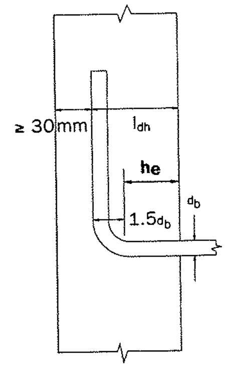
$\xi_R = \varphi_{CR}\varphi_{sx}\varphi_{sy}\varphi_{cx}\varphi_{cy}$ - overall reduction factor

Where: $\varphi_{CR} = 0.75$ for cone area in cracked section
= 1.00 in any other case.
φ_{sx} & φ_{sy} = spacing reduction factor
$$\varphi_{sx} = (1 + (n_x - 1)(S_x / S_{CR}))/n_x \le 1$$
n_x = number of anchors in x direction
S_x = centre to centre spacing in x direction
$S_{CR} = 3h_e$
φ_{sy} = as for φ_{sx} with subscript y in place of x.
φ_{cx} & φ_{cy} = edge reduction factors
$$\varphi_{cx} = \{0.3 + (0.7cx / 1.5h_e)\} \le 1$$
cy = edge distance in y direction
φ_{cy} = as for φ_{cx} with subscript x in place of y

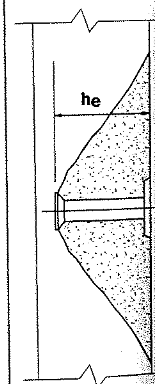
is $f_s < f_y$

yes / no → Use normal limit state for design of joint capacity

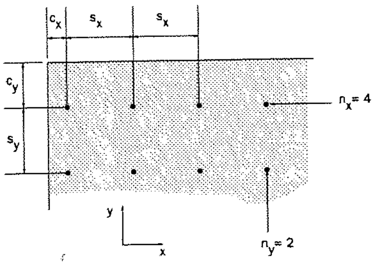
no / yes → Use elastic design of joint capacity



Hooked Bar effective depth h_e



Threaded Insert effective depth he



Definition of Spacing Parameters used by the ψ-method.

Example calculation of typical base fixing for Threaded Insert Connection and Hooked Bar Connection

Material Properties $MPa := 10^6 Pa$ $kN := 10^3 N$ $kNm := kN \cdot m$ $GPa := 10^9 \cdot Pa$

$f_c := 30MPa$ $f_y := 500MPa$ $\phi := 0.85$

Concrete modulus of Elasticity $E_c := \left(3320 \cdot \sqrt{MPa} \cdot \sqrt{f_c} + 6900MPa\right)$ Steel Modulus of Elasticity $E_s := 200GPa$

Wall Panel Properties

Wall thickness
$wall_t := 120mm$

Unit width of
$B := 1000mm$

Wall panel flexural strength

Reinforcing - bar diameter $d_r := 12mm$ centres $c_r := 300mm$ top cover

$wall_t$ d_r

Area of reinforcing per unit length

$$A_s := \frac{\pi \cdot d_r^2}{4} \cdot \frac{B}{c_r} \qquad A_s = 377\,mm^2$$

$$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot B} \qquad a = 7.392\,mm \qquad \phi M_n := \phi \cdot A_s \cdot f_y \cdot \left(\frac{wall_t}{2} - \frac{a}{2}\right) \qquad \phi M_n = 9\,kNm$$

Nominal tensile capacity of section $M_t := 0.6 \cdot \sqrt{f_c} \cdot \frac{B \cdot wall_t^2}{6} \cdot MPa^{0.5}$ $M_t = 7.9\,kNm$

Over strength - cl. 2.6.5.5 (b) iii)

$$a_{com} := \frac{A_s \cdot 1.35 \cdot f_y}{0.85 \cdot \left(f_c + 15MPa\right) \cdot B} \qquad a_{com} = 6.65\,mm$$

$$\phi Mover_{ncom} := \left(A_s \cdot 1.35 f_y\right) \cdot \left(\frac{wall_t}{2} - \frac{a_{com}}{2}\right) \qquad \boxed{\phi Mover_{ncom} = 14.4\,kNm}$$

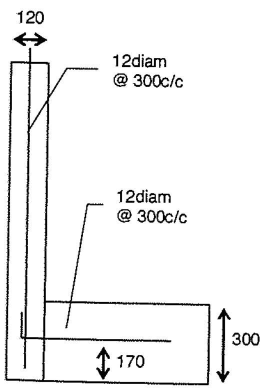
Overstrength factor $\frac{\phi Mover_{ncom}}{\phi M_n} = 1.6$

Foundation to wall panel connection:

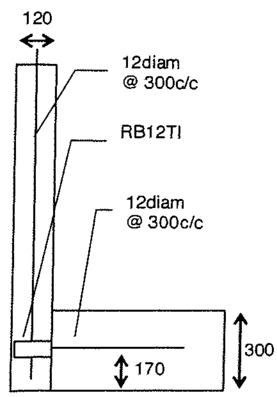
Foundation depth
$D := 300mm$

Starter bars/ inserts of diameter $d_b := 12mm$ at spacing of
$s := 300mm$

Hooked bar pullout cone capacity



Threaded Insert pullout cone capacity



REIDBAR & FITTINGS

Effective embedment depth of hooked bar with cover to starter = 30mm

Bar height $ht_{bar} := 170mm$

$$h_{e\_bar} := wall_t - \frac{3 \cdot d_b}{2} - 30mm \qquad h_{e\_bar} = 72\,mm$$

Reduction factor for cracked section

$\psi_{cr\_bar} := 0.75$

Critical Spacing for embedment depth

$s_{cr\_bar} := 3 \cdot h_{e\_bar} \qquad s_{cr\_bar} = 216\,mm$

Spacing reduction factors therefore are

$\psi_{sx\_bar} := 1 \qquad \psi_{sy\_bar} := 1$

Edge reduction factors are

$$\psi_{cy\_bar} := 0.3 + 0.7 \cdot \frac{ht_{bar}}{1.5 \cdot h_{e\_bar}}$$

$\psi_{cy\_bar} = 1.402 \qquad \psi_{cx\_bar} := 1$

so $\psi_{cy\_bar} := 1$

Total reduction factors to apply

$$\xi_{R\_bar} := \psi_{cr\_bar} \cdot \psi_{sx\_bar} \cdot \psi_{sy\_bar} \cdot \psi_{cy\_bar} \cdot \psi_{cx\_bar}$$

$$\xi_{R\_bar} = 0.75$$

$$d'_b := \frac{d_b}{mm} \qquad h'_e := \frac{h_{e\_bar}}{mm} \qquad f_c := \frac{f'_c}{MPa}$$

$$f_{s\_bar} := \left(\frac{h'_e}{0.23 \cdot d'_b{}^{\frac{4}{3}}} \right)^{1.5} \cdot \sqrt{f_c} \cdot \xi_{R\_bar}$$

$f_{s\_bar} := f_{s\_bar} \cdot MPa \qquad f_{s\_bar} = 158\,MPa$

Yielding of the reinforcement cannot be achieved before pullout failure will occur.
Using elastic analysis for an opening moment on the connection:

Modular ratio

$$m' := \frac{E_s}{E_c} \qquad m' = 7.973$$

$$d'_1 := ht_{bar} \qquad A'_{st} := \pi \cdot \frac{d_b^2}{4} \cdot \frac{B}{s} \qquad A'_{st} = 377\,mm^2$$

$na' := 100mm$

Effective embedment depth of threaded insert with cover to insert = 12mm (galvanised)

Insert height $ht_{insert} := 170mm$

$h_{e\_insert} := 108mm \qquad h_{e\_insert} = 108\,mm$

Reduction factor for cracked section

$\psi_{cr\_insert} := 0.75$

Critical Spacing for embedment depth

$s_{cr\_insert} := 3 \cdot h_{e\_insert} \qquad s_{cr\_insert} = 324\,mm$

Spacing reduction factors therefore are

$$\psi_{sx\_insert} := \frac{s}{s_{cr\_insert}} \qquad \psi_{sy\_insert} := 1$$

Edge reduction factors are

$$\psi_{cy\_insert} := 0.3 + 0.7 \cdot \frac{ht_{insert}}{1.5 \cdot h_{e\_insert}}$$

$\psi_{cy\_insert} = 1.035 \qquad \psi_{cx\_insert} := 1$

so

$\psi_{cy\_insert} := 1$

Total reduction factors to apply

$$\xi_{R\_insert} := \psi_{cr\_insert} \cdot \psi_{sx\_insert} \cdot \psi_{sy\_insert} \cdot \psi_{cy\_insert} \cdot \psi_{cx\_insert}$$

$$\xi_{R\_insert} = 0.69$$

$$d'_b := \frac{d_b}{mm} \qquad h_e := \frac{h_{e\_insert}}{mm} \qquad f_c := \frac{f'_c}{MPa}$$

$$f_{s\_insert} := \left(\frac{h_e}{0.23 \cdot d'_b{}^{\frac{4}{3}}} \right)^{1.5} \cdot \sqrt{f_c} \cdot \xi_{R\_insert}$$

$f_{s\_insert} := f_{s\_insert} \cdot MPa \qquad f_{s\_insert} = 268.8\,MPa$

Yielding of the reinforcement cannot be achieved before pullout failure will occur.
Using elastic analysis for an opening moment on the connection:

Modular ratio

$$m := \frac{E_s}{E_c} \qquad m = 7.973$$

$$d_1 := ht_{insert} \qquad A_{st} := \pi \cdot \frac{d_b^2}{4} \cdot \frac{B}{s} \qquad A_{st} = 377\,mm^2$$

Given

$$\left[na'^2 - \frac{m' \cdot A'_{st} \cdot 2}{B} \cdot (d'_1 - na')\right] = 0$$

$d'_{na} := \mathrm{Find}(na')$ $d'_{na} = 29.1\,\mathrm{mm}$

stress in the steel is $f_{s\_bar} = 158\,\mathrm{MPa}$

Total force in reinforcing bars is $F_{s\_bar} := f_{s\_bar} \cdot A'_{st}$

moment will be $M_{e\_bar} := F_{s\_bar} \cdot \left(d'_1 - \frac{d'_{na}}{3}\right)$

Hooked bar opening moment capacity:

$M_{e\_bar} = 9.5\,\mathrm{kNm}$

$na := 100\mathrm{mm}$

Given

$$\left[na^2 - \frac{m \cdot A_{st} \cdot 2}{B} \cdot (d_1 - na)\right] = 0$$

$d_{na} := \mathrm{Find}(na)$ $d_{na} = 29.1\,\mathrm{mm}$

stress in the steel is $f_{s\_insert} = 268.8\,\mathrm{MPa}$

Total force in reinforcing bars is $F_{s\_insert} := f_{s\_insert} \cdot A_{st}$

moment will be $M_{e\_insert} := F_{s\_insert} \cdot \left(d_1 - \frac{d_{na}}{3}\right)$

Reid Threaded Insert opening moment capacity:

$M_{e\_insert} = 16.2\,\mathrm{kNm}$

Conclusions:

The **hooked bar** base connection is an unsafe design with brittle failure of the connection likely to occur before the yielding of the wall panel. $M_{e\_bar} = 9.5\,\mathrm{kNm}$ compared to possible wall strength of $\phi Mover_{ncom} = 14.4\,\mathrm{kNm}$

On the otherhand the **Reid threaded Insert** connection is safe because yielding in the wall panel is likely to occur before cone failure in the foundation connection.
$M_{e\_insert} = 16.2\,\mathrm{kNm}$ compared to possible wall strength of $\phi Mover_{ncom} = 14.4\,\mathrm{kNm}$

NOTE: Software for wall/base calculation is available on Reids Resource Disc or direct from Reids Engineering Manager Ph 09 920 4346

REFERENCES

1) NZS:3101:Part 1:1995 Concrete Structures Standard The Design of Concrete Structures
2) NZS:3101:Part 2:1995 Commentary on The Design of Concrete Structures
3) NZS:3101:Part 2:1995 Amendment No1 December 1998
4) Tensile Capacity of Steel Connectors with Short Embedment Lengths in Concrete - Restrapo- Prosada and Park August 1993
5) Tensile Capacity of Hooked Bar Anchorages with Short Embedment Lengths in Concrete - Nigel Watts University of Canterbury September 1996
6) Methods of Joining Precast Concrete components to form Structural Walls- Maureen Ma University of Auckland 1999
7) The Design and Construction of Tilt-up Reinforced Concrete Buildings - Restrepo, Crisafulli and Park. University of Canterbury 1996
8) The Performance of Reidbar Couplers in Seismic Resistant Frame Structures - Bassim Bahr-Aliloom University of Auckland Feb 1997
9) Assessing the Seismic Performance of Reinforcement Coupler Systems - Anselmo Bai University of Auckland 2003
10) Tensile Capacity of Headed Anchors with Short Embedment Lengths in Concrete - Barry Magee University of Canterbury September 1996

REIDBAR COMPONENTS

REIDBAR™ THREADED INSERTS

Reidbar Threaded Inserts

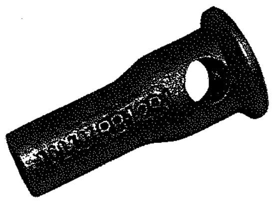
Designed for casting into concrete and provide fixing points for attachments of starter bars and structural members to be bolted directly to the concrete structure.

Part No.	Bar dia. Ø	Overall length	Foot dia.	Qty (per box)
RB12TI	RB12	100mm	38mm	75
RBA16TI	RB16	118mm	50mm	30
RB20TI	RB20	148mm	64mm	25
RB25TI	RB25	191mm	80mm	10
RB32TI	RB32	210mm	101mm	1
Galvanised				
RB12TIG	RB12	100mm	38mm	75
RBA16TIG	RB16	118mm	50mm	30
RB20TIG	RB20	148mm	64mm	25
RB25TIG	RB25	191mm	80mm	10
RB32TIG	RB32	210mm	101mm	1

Reid are pleased to add the 32mm size Reidbar Threaded Insert to our range.

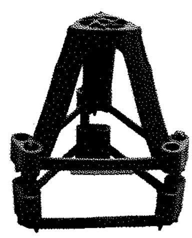
Reidbar Reduced Threaded Insert

Designed for use in thin, specialty panels under 100mm thick.



Part No.	Bar dia. Ø	Overall length	Foot dia.	Qty (per box)
RB12RI	RB12	78mm	38.75mm	60

Reidbar Threaded Insert Chair



Part No.	Description	Qty (per box)
TICHAIR	Threaded Insert Chair	90

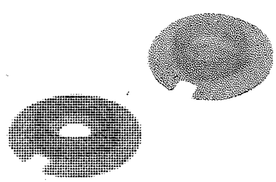
Reidbar Plastic Nail Plates

Part No.	Description	Qty (per box)
NP12RB	Nail Plate to suit Reidbar™ RB12 Threaded Inserts	100
NP16RB	Nail Plate to suit Reidbar™ RB16 Threaded Inserts	100
NP20RB	Nail Plate to suit Reidbar™ RB20 Threaded Inserts	100
NP25RB	Nail Plate to suit Reidbar™ RB25 Threaded Inserts	100
NP32RB	Nail Plate to suit Reidbar™ RB32 Threaded Inserts	100

NEW SIZE!

32mm Nail Plate is a welcome addition to the Reidbar range of accessories.

Reinforcing Grommet



Part No.	Inside dia. Ø	Outside dia. Ø	Bar dia. Ø	Qty (per box)
RG12*	33mm	67mm	RB12	100
RG16*	33mm	67mm	RB16	100
RGB	33mm	67mm	Various	100

* Indent only

7.3.14.2 * REFER TO AMENDMENT #1.
The development length, L_{dh}, for hooks shall be computed as follows:

$$L_{dh} = 0.24\alpha_b\alpha_1\alpha_2\frac{f_y d_b}{\sqrt{f'_c}} \geq 8d_b \text{ or } \cancel{150} \text{ mm} \quad \text{(Eq. 7-11)}$$

The value of f'_c used in equation 7-11 shall not exceed 70 MPa

Ldh = 200

and where

(a) α_b is given by 7.3.7.3(a);

(b) For 32 mm bars or smaller with side cover normal to the plane of the hooked bar not less than 60 mm and cover on the tail extension of 90° hooks not less than 40 mm

$\alpha_1 = 0.7$

and in all other cases $\alpha_1 = 1.0$

(c) For confinement by closed stirrups or hoops spacing of 6 d_b or less, where

$$\frac{A_{tr}}{s} \geq \frac{A_b}{1000}$$

$\alpha_2 = 0.8$

and in all other cases $\alpha_2 = 1.0$

7.3.14.3
Hooks shall not be considered effective in developing reinforcement in compression.

7.3.15 *Mechanical anchorage*

7.3.15.1
Any mechanical device capable of developing the design force in the reinforcement may be used as anchorage.

7.3.15.2
The adequacy of such mechanical devices shall be established by rational design or suitable tests.

7.3.16 *Splices in reinforcement – General*

7.3.16.1
Splices of reinforcement shall be made only as required or permitted on the design drawings or in specifications. Except as provided herein, all welding shall conform to NZS 4702.

7.3.16.2
In the design and execution of welding of reinforcing bar manufactured to NZS 3402, appropriate account shall be taken of the process of manufacture. Reinforcing bars not conforming to NZS 3402 shall not be welded, unless the nominated welding technique has been demonstrated by tests to produce welds in the specific local conditions that have the required mechanical and metallurgical properties.

Welds in reinforcing bars shall not be made less than 3 d_b from the commencement of bends or that part of a bar which has been bent and re-straightened.

magnitude of compressive stresses on the inside of the hook. Only standard ACI hooked bars were tested and the influence of larger radius of bend was not evaluated. The test results indicate that as the straight lead length increases, the lateral splitting force which develops in the side cover decreases; this is reflected in an improvement in hook capacity.

The recommended provisions include adjustments to reflect the resistance to splitting provided by enclosure in transverse reinforcement. If the side cover is large so that side splitting is effectively eliminated, as in mass concrete, the product of the factors α_b, α_1 and α_2 as given in 7.3.14.2 may be used. Minimum values of L_{dh} are indicated to prevent failure by direct pullout in cases where the standard hook may be located very near the critical section. No distinction is made between top bars and other bars.

In many cases where the value of L_{dh} given by equation 7-11 is used, the value of d_i required will be greater than that given in table 7.1 as it will be governed by equation 7-1. In such cases, if it is desired to reduce the value of d_i to that given in table 7.1, the value of L_{dh} will have to be increased above that given to be used by equation 7-11 in order to give an increased value for the lead length L_b as shown in figure C7.1 which will allow a reduced value of d_i from equation 7-1.

C7.3.15 *Mechanical anchorage*

Mechanical end anchorages should be made adequate for strength both for prestressing tendons and for reinforcing bars .

C7.3.16 *Splices in reinforcement – General*

For ductility of a member, lap splices should be adequate to develop more than the yield strength of the reinforcement; otherwise a member may be subject to sudden splice failure when the yield strength of the reinforcement is reached. The lap splice lengths specified in the Standard satisfy this ductility requirement for members.

Splices should, if possible, be located away from points of maximum tensile stress.

C7.3.16.1

Due to the low carbon metallurgy of reinforcing steel manufactured to NZS 3402, the steel is considered readily weldable. However, NZS 3402 permits a range of manufacturing processes for the production of steel reinforcement. Due care must be exercised for welding of such reinforcement because the welding process can alter the metallurgy and microstructure of the as-rolled bars. In certain situations this may result in lower yield strengths and lower ultimate tensile strengths in the heat affected zones of the welds sites. This may lead to detrimental behaviour with loss of ductility in the bar and fracture of the bar may occur. Refer to NZS 4702 and the reinforcement manufacturer's recommendations for details of appropriate welding techniques.

C7.3.16.2

Reinforcing steels not conforming to NZS 3402 will require different welding techniques and the designer and fabricator must become familiar with these techniques before designing a weld or attempting to weld the steel.

C7.3.16.3

Research on lap splices with bars of diameter greater than 40 mm is limited. There is insufficient data to establish lap lengths for either tensile or compressive lap splices for these bars.

C7.3.16.4

The increased length of lap required for bars in bundles is based on the reduction in the exposed perimeter of the bars. Where the factors in this clause are applied it is not intended that the factors in 7.3.11 should also be applied.

C7.3.16.5

Welds complying with 7.3.16.5(a) can withstand the most severe strain or stress cycles. Hence they are acceptable in all locations, in particular, for splicing main longitudinal reinforcement in plastic hinge regions and in beam-column joints. The appropriate weld quality in the NZS 4702 classification[7.13] would be S.

7.1 Notation (page 72)

Under the symbol for A_{tr} delete "$A_{tr} = A_t$" and substitute "$A_{tr} = A_t$ when $n \leq 6$."

(Amendment No. 1, December 1998)

7.1 Notation (page 73)
Delete definition of n_L.

(Amendment No. 1, December 1998)

7.3.5.5 (page 75)
In lines 1 and 2 **delete** "not further apart than 2 times the wall or slab thickness" and **substitute** "not further apart than 3 times the wall or slab thickness".

(Amendment No. 1, December 1998)

7.3.7.3(c) (page 77)
Delete equation 7-5 and **substitute**:

$$\text{"} \alpha_d = 1 + \sqrt{\left(\frac{A_{tr}}{s}\right)\left(\frac{f_{yt}}{80nd_b}\right)} \quad \text{......} \quad \text{(Eq. 7-5)"}$$

(Amendment No. 1, December 1998)

7.3.9.3 (page 77)

Delete "$\frac{A_{tr}}{s} \geq \frac{A_s}{600}$" and **substitute** "$\frac{A_{tr}}{s} \geq \frac{A_b}{600}$".

(Amendment No. 1, December 1998)

7.3.14.2 (page 79)
Delete the words "or 150 mm" .... from the (Eq. 7-11).

Add 2 new paragraphs after the line containing .... (Eq. 7-11).

"The development length, L_{dh}, determined from equation 7-11 shall apply when there is no likelihood of a failure mode of the pull-out of a cone of concrete from the volume of concrete in which the bar is anchored.

If a cone of concrete pull-out is likely then a rational analysis or suitable testing shall account for the effects of the proximity of the anchored bars to other loaded embedded items and to the edges of elements."

(Amendment No. 1, December 1998)

Add a new clause (page 80)

7.3.16.7
For bars spliced by lapping that involves offsetting of the bars by cranking, the slope of the inclined portion of bars with respect to the axis of the longitudinal bars shall not exceed 1 in 6. Transverse reinforcement or some other means of restraint at a bend in the cranked bar shall be provided to carry 1.5 times the transverse thrust that results because of the inclination of the bar. The magnitude of the transverse thrust shall be determined assuming that the bar is stressed to f_y. The resultant restraint force, provided by ties, spirals or other means of restraint, shall act through the centre of the bend of the cranked bar or bars.

(Amendment No. 1, December 1998)

(i) The spacing between that bar and any adjacent bar or fixing loaded in a similar direction is greater than or equal to three times d_b over L_{dh} for that bar; and

(ii) The distance normal to the axis of the bar to the side or edge of the element is greater than or equal to two times d_b over L_{dh} for that bar.

$$L_{dh} = 0.24\alpha_b \alpha_1 \alpha_2 \frac{f_y d_b}{\sqrt{f'_c}} \geq 8d_b \qquad \text{(Eq. 8–12)}$$

where

f'_c shall not be taken greater than 70 MPa

α_b is given by 8.6.3.3 (a)

α_1 = 0.7 for 32 mm bars or smaller with side cover normal to the plane of the hook ≥ 60 mm, and cover on the tail extension of 90° hooks equal to or greater than 40 mm

= 1.0 for all other cases

α_2 = 0.8 where confined by closed stirrups or hoops spaced at $6d_b$ or less and which satisfy the relationship $\frac{A_{tr}}{s} \geq \frac{A_b}{1000}$

= 1.0 for all other cases

8.6.10.3.2 *Determination of development length where not covered by 8.6.10.3.1*

For situations other than as described by 8.6.10.3.1(a) and (b), the development length of a hook shall be determined from a rational analysis or suitable testing that takes into account the effects of the proximity of the anchored bar to edges of elements and to other loaded embedded items.

8.6.10.3.3 *Development length of standard hooks anchoring around transverse bars*

The development length L_{dh} of a deformed bar terminating in a standard hook as determined from 8.6.10.3 may be reduced by 20 %, provided that two transverse bars having a diameter equal to or larger than that of the bent bar are placed in contact with the inside of the bend and extend for a distance equal to or greater than $3d_b$ beyond the centreline of the bent bar.

8.6.10.4 *Hooks in compression*

Hooks shall not be considered effective in developing reinforcement in compression.

8.6.11 *Mechanical anchorage*

8.6.11.1 *General*

Any mechanical device used alone as an anchorage, or used in combination with an embedment length beyond the point of maximum stress in the bar, shall be capable of developing the upper characteristic breaking strength of the reinforcing bar without damage to the concrete or overall deformation of the anchorage.

8.6.11.2 *Upper bound breaking strength for the reinforcing bar – definition*

The upper characteristic breaking strength of the reinforcing bar may be derived from 1.15 times the upper characteristic yield strength specified by AS/NZS 4671, or otherwise shall be determined from an appropriate testing programme.

8.6.11.3 *Adequacy of mechanical devices*

Mechanical anchorage systems relying on interconnecting threads or mechanical interlock with the bar deformations for attachment of the anchorage to the bar shall meet both the permanent extension and fatigue strength criteria of 8.7.5.2.

C8.6.10.3 *Development length of standard hooks in tension*

The required development length L_{dh} for hooked bars in tension in accordance with 8.6.10, may be larger than what might be available in a column when the requirements of 9.4.3.2 shown in Figure C9.18 are to be satisfied. In such situations it is better to improve the bearing conditions in the bend than to provide extra straight anchorage length beyond the 90° bend. When transverse bars, as shown in Figure C8.1, are provided, a 20 % reduction in the development length L_{dh} of Figure 8.1 may be made. When beam bars are anchored within column bars in the core of a beam column joint, the application of the multiplier α_1 = 0.7 in 8.6.10.3(b) is appropriate.

The bars placed in the bend help reduce the local bearing stresses and reduce the tendency for splitting cracks to form in the plane of the bend. The extension of these bars by $3d_b$ beyond the plane of the bar does not imply any limit on the spacing of adjacent bent bars.

When the same bar is required to develop yield strength in compression, the bent portion of the anchorage must be disregarded in satisfying the requirements of 8.6.5.1. However, when bars are anchored in column cores, as described above, the confinement may be considered to be equivalent to that implied in 8.6.5.3. The development of bars in compression will commence closer to the inner face of exterior columns.

C8.6.11 ***Mechanical anchorage***

Mechanical end anchorages should be made adequate for strength both for prestressing tendons and for reinforcing bars.

C5 Basis of Design and Calculations for Representative Structural Elements

C5.1 INTRODUCTION

C5.1.1 In this appendix, I have indicated what I consider to be the appropriate methodology to be used when designing connections between precast concrete panels, cantilevering vertically, and the foundations which are intended to stabilise them.

C5.1.2 I have included some representative calculations on the above for two representative buildings in order to illustrate the approach to be used.

C5.2 DESIGN APPROACH

C5.2.1 Precast panel/foundation base connection using "TCM" connectors

(a) Acknowledging the flexibility of the superstructure, precast panels are designed to cantilever vertically from foundation level under maximum applied face load, often irrespective of whether or not horizontal propping is provided by transom beams at the top level.

(b) Obviously the requirement for base stability under face loading resistance will be fully mobilised for the fire burnout condition, for which a specified face load of 0.5 kPa is applied.

C5.2.2 For a range of reasons, the designer(s) in this project has used proprietary "TCM" connection reinforcing splices cast into the cover concrete of the precast wall panels to provide the reinforcing continuity between the wall panel and the stabilising foundation element.

NB : The industry reference commonly used to justify this detail is to Reid Systems Manual at p112, a copy of which is included in Appendix B.

C5.2.3 For review purposes, I believe the dependable moment strength of this connection can be demonstrated when the dependable strength of the TCM connectors used can resist the lesser of :

(a) tension and/or shear demand derived from flexural yield of the vertical reinforcing in the panel cantilevering above the slab under face loads (including appropriate allowances for overstrength); or

(b) tension and/or shear demand derived from elastic response of the panel assuming elastic response as prescribed by the relevant loading standard for $\mu = 1$ and $S_p = 1$.

This means that, in practice, the demand calculated for $\mu = 1.25$, $S_p = 0.67$ within the relevant loading standard needs to be doubled when calculating the loads to be resisted by the TCMs in the wall/foundation base connection, and then margins included to ensure the anchorage remains elastic.

C5.2.4 The shear demand used for the above capacity checks should include those derived from a reasonable estimate of panel uplift or sliding under the following demand scenarios -

(a) Calculated via plane shear in the panel

(b) An oblique (45°) earthquake action on the building as a whole, and on the panel itself

NB : Whilst these thresholds are demanding, they are appropriate given the acknowledged lack of ductility in the TCM connection used to stabilise the panels in this situation, and the need to ensure elastic behaviour is sustained within the connection itself whilst capacity-based demand is being applied within the wall panel itself.

C5.2.5 Representative calculations have been prepared separately for two of the subject buildings and are included in Appendix C4 to address the particulars referred to in the above analysis.

C5.3 DESIGN RULES FOR EMBEDMENTS

C5.3.1 Whilst the 1998 amendment to NZS 3101:1995 cl 7.3.14 permitted incorporation of TCM-type embedments within foundation base connections in precast concrete wall panels, the design methodology used for these connections was largely left to industry to prescribe. These provisions were not fully codified until the publication of the NZS 3101:2006 standard (with section 17 covering embedded items, fixings, and secondary structural elements) and its subsequent citation as part of B1/VM1 in December 2008.

C5.3.2 In the interim, viz before December 2008, standard industry practice was for designer to refer to the relevant provisions of the "Precast Concrete Guidelines" (refs R1) which recommended essentially the same approach.

C5.4 RELEVANT INDUSTRY STANDARDS

C5.4.1 Key points from Reid Construction Systems Manual 2007 (p112-115)

(a) Conventional bar anchorage using standard bend detail, as per NZS 3101:2006 cl 7.3.14.2*.

> The minimum development length of 150 mm is removed from clause 7.3.14.2 and two new clauses added
>
> *"The development length, L_d , calculated using eqn 7-11 shall apply when there is no likelihood of a failure mode of a pullout of a concrete cone from the volume of concrete in which the bar is anchored"*
>
> *"If a cone of concrete pullout is likely then a rational analysis* or suitable testing shall account for the proximity of the anchored bars to other loaded elements and to edges of elements"
>
> Essentially these amendments say that one should not use hooked bars to develop full bar capacity **unless concrete cone pullout capacity exceeds the bar strength.**

NB : Reference error noted - the correct NZS 3101:2006 clause reference is 8.6.11.

(b) This gives the minimum wall thickness for use of conventional standard bend anchorage vertically within a precast wall panel as :

(i) HD12 = 214 mm; and

(ii) HD16 = 275 mm

- which exceeds the thicknesses typically used for cantilevered wall panels.

NB : Manual assumes (f'_c = 30 MPa; f_y = 500 MPa) for this calculation.

C5.4.2 An alternative anchorage of reinforcing into the cover zone of centrally reinforced precast concrete panels using Reid threaded inserts permits lesser wall thicknesses as :

(a) RB12TI - minimum wall thickness 120 mm; and

(b) RB16TI - minimum wall thickness 140 mm.

C5.5 SESOC "PRACTICE NOTE" RECOMMENDATIONS FOLLOWING CANTERBURY EARTHQUAKE

C5.5.1 In December 2011, following the Canterbury earthquake, the NZ Structural Engineering Society (SESOC) published some "interim design standards" practice notes entitled "*Design of Conventional Structural Systems Following the Canterbury Earthquakes*" (ref R10), within which section 3 "Concrete Walls" (p9-17) are relevant to this discussion.

C5.5.2 Particular recommendations from this interim SESOC practice note which relate to tilt up precast wall construction includes the following.

(a) cl 3.1 Singly Reinforced Walls (p9)

Recommendation is that singly reinforced walls should be designed for nominally ductile (μ = 1.25) actions. Typically provide closed stirrup cages at each end of a wall segment to confine the anchorage of the horizontal reinforcing steel (refer to Figure 2) unless $V^*<øV_c$, or $\varepsilon_c<0.001$.

(b) cl 3.3 Wall Thicknesses (p11)

NZS 3101:2006 cl 11.3.11.2 prescribes minimum wall thickness in terms of reinforcing bar size, eg $t_w \ngtr 7d_b$, for nominally ductile behaviour, and lesser for ductile and limited ductile repairs (refer Table 1) plus Notes 1 3.

(c) cl 3.4 Local Bar Buckling (p11-12)

Refers to requirement for confining steel should be provided in all walls (ductile or nominally ductile) where NZS 3101:2006 cl 11.3.11.5 SESOC Practice Note recommends that "full anti-buckling and confinement reinforcing (ref cl 11.4.6) over full length of the compression zone, unless it can be shown that the wall has sufficient capacity to resist 1.5 times the ULS forces without yielding any bars".

(d) cl 3.5 Global Wall Buckling (p13)

Refers to limiting height/thickness (h/t) ratios for eg singly reinforced walls ($h/t<20$), above which slenderness effects need to be considered.

Code requirement is, for outside plastic hinge regions for slender walls (*h/t>20*), wall buckling should be checked in accordance with clauses 11.3.5, 11.3.6, and 11.3.7. In yielding regions wall buckling should be checked in accordance with clause 11.4.2.

(e) cl 3.6 Minimum Reinforcement (p14)

Recommends minimum reinforcement content calculation made to comply with NZS 3101:2006 cl 11.3.11.3 shall comply with a conservatively framed formula based on upper bound concrete strengths based on $f'_c = 2 \times f'_c$.

(f) cl 3.7 Distribution of Reinforcing Steel (p15)

Recommendation is that, to limit the development of tensile strains on the ends of flexurally-dominated walls, reinforcing should be lumped at the ends of a wall, with minimum reinforcing distributed along the web.

(g) cl 3.8 Precast Panel Splices (p15-17)

SESOC recommends that, in order to concentrate tension force at panel/floor splice zones, where excessive strains develop in the vertical reinforcing bars at the end of the panel, the splice bar within the Drossbach duct is debonded in the anchorage zone below the joint, with the debond length calculated using an empirical formula of $L_{dh} = \Delta_w \frac{L_w}{0.05h_w}$.

SESOC recommendation is that "precast panel splices must allow for the debonding of reinforcement where yielding is expected" and "Drossbach ducts must be fully confined".

(h) cl 3.9 Precast Panel Embedded Anchors (p17)

SESOC recommends that shallow embedded connections shall not be used for primary structural load paths.

(i) cl 3.10 Compatibility Effects in Gravity Structure (p17)

SESOC recommends that all gravity frames and members in wall structures shall be detailed to accommodate $1.5/S_p$ times the ULS drifts. This may be achieved by detailing the gravity columns (or other support) for ductility in accordance with the seismic design provisions of the relevant materials standard, for both (column) confinement and shear.

C5.6 REVIEW OF KOA CALCULATIONS SECTION C COVERING THE WALL PANEL/FOUNDATION CONNECTION ON THE CARPET COURT BUILDING (ref 49158/4 in Appendix B4)

C5.6.1 In this section I consider the source of the under-capacity in the panel/foundation connection referred to at item 6.2 in the review assessment for the building given in Appendix B4.

C5.6.2 In my opinion, the designer has assumed that the overturning moment in the slab can be resisted by a force couple comprising the cast-in insert and a reaction point at the base of the panel some 300 mm below (refer KOA calculations at pages C3 (sketch), C6 (ht = 300 mm), and C7 (M insert)). This assumption would be valid if the edge thickening of the slab was 400 mm deep (reference Section 1/S01 on drawing S12) for the full length of the wall panel, but does not, in this case, because this detail only occurs at the pile support. In reality, the base of the panel is unrestrained as per Section 6/S01 on drawings S13 and the 300 mm lever arm cannot apply.

C5.6.3 By my assessment, the actual level arm to the TCM connection can only be around 100 mm, so that the resisting moment available can only be around 8 kN-m/m or around 30% of what is required.

As noted in Appendix C5 of this report, the foundation/wall panel connection incorporating the TCM anchorage must be designed at least to resist the overstrength moment in the wall panel, which is demonstrably not the situation with the details proposed for use in this building.

C5.6.4 As part of my review, I have prepared some preliminary design calculations for the TCM inserts intended to reinforce the wall panel/foundation junction for the resistance of both out-of-plane face/bending loads and in-plane (shear) loads in a typical panel within this building.

C5.6.5 These calculations suggest that these TCM connections in this particular building are significantly under-designed - ie say 20% of a fully compliant requirement - for the critical severe seismic (ULS) design case required by the relevant loading standard - refer Appendix C5 calculations ref 49158/4/01-05.

Appendix C5.7

Representative calculations for the wall panel/foundation "TCM" connection

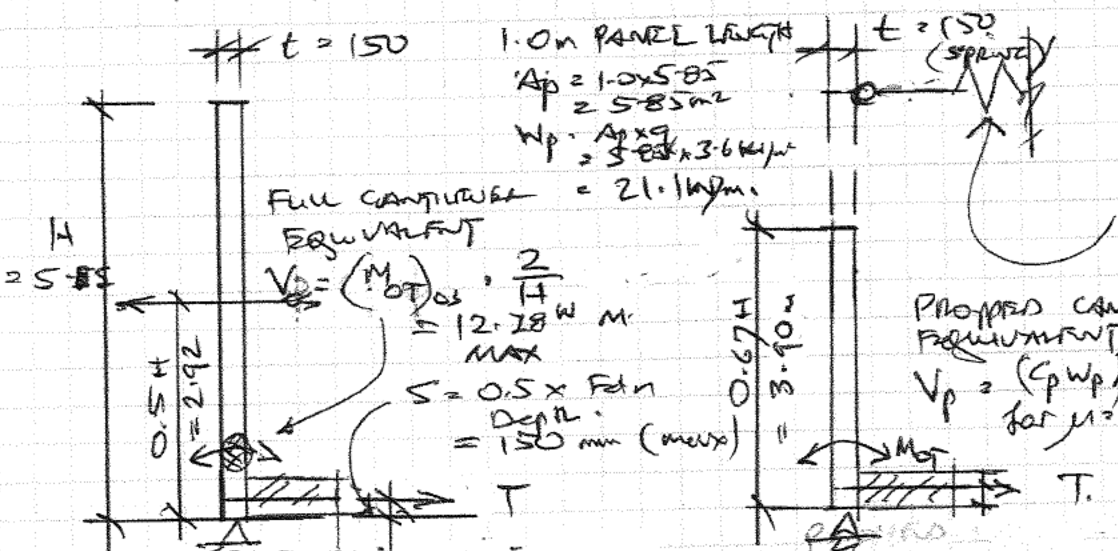
49158/1 - 408 Queen St

CLIENT: IPENZ / MASTERGON BUILDINGS
JOB: 408 Queen St
REF 102610
CLIENT REF. NO:
BY: B.J. BROWN
CHECKED:

Fraser Thomas
• CONSULTING ENGINEERS
• RESOURCE MANAGERS
• ENVIRONMENTAL CONSULTANTS
• SURVEYORS & PLANNERS

JOB No/PAGE 49158/1/01
DATE 15/5/17

GEOMETRY FOR WALL PANEL / FOUNDATION JUNCTION FOR CHECKING FACE LOAD RESISTANCE



PROPPING FROM PORTAL TRANSOM WITH LOW EQUIVALENT SPRING STIFFNESS

FULL CANTILEVER STABILITY UNDER ϕM_{OS} CASE

$[M_{OT}]_{OS} = 1.35 A_s f_y j d$

$M_{res} = T \times S$

$\therefore T = \frac{[M_{OT}]_{OS}}{S}$

For H = 5.85m

t = 150mm

As (HD12–150) = 747mm²/m

$\phi M_n = \phi A_s f_y j d$ dependable

$= 0.9 \times 747 \times 500 \times 0.95 \times 75$

$= 23.95 kNm/m$

$M_{OS} = \frac{1.35}{0.9} \times 23.95 kNm/m$ @ FLEXURAL OVERSTRENGTH

$= 35.92 kNm/m$

FDN DEPTH
D = 250
S = 100 (closing)
150 (opening)

FULL CANTILEVER INSERT TCM DEMAND:-

$T_0 = \frac{M_{OS}}{S} = \frac{35.92 kNm/m}{0.10 m} = 359.2 kN/m$ (closing)

$= 359.2 kN/m \times 0.15m$

$= 53.88 kN/TCM$

PROPPED CANTILEVER STABILITY UNDER NZS 4203:1992 Part 4
μ=1.0, Sp=1.0

$\rightarrow C_h(T) = 0.959$. (T<0.4

$M_{OT} = C_h(T) W_p \times \frac{2H}{3}$

$= 0.959 \times [21.1 kN/m \times \frac{2}{3}] \times \frac{H}{3}$ 14.07

$= 13.49 kNm$

ie 37.6% of M_{OS} for FULL CANTILEVER

PROPPED CANTILEVER INSERT TCM DEMAND

$T_0 = \frac{M_{OT} (\mu=1, Sp=1.0)}{S}$

$= \frac{13.49}{0.1}$

$= 134.9 kN/m$ CLOSING

$= 20.23 kN/TCM$

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JOB: 408 Queen St

CLIENT REF. NO:

BY: B. J. BROWN

CHECKED: 239.5 kN/m × 0.15m = 35.93 kNm/m

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DATE: 15/5/17

$$T_C = \frac{M_{os}}{S} = \frac{35.92}{0.15} = 239.5 \text{kN/m}$$

OPENING

FOUNDATION RESISTANCE TO PANEL OVERTURN

DH10 @ 150 IN 250 THICK FDN

CLOSURE MOMENT:	OPENING MOMENT:
$d = 100$	$d = 150$
$A_s = 747 \text{mm}^2/\text{m}$	$A_s = 747 \text{mm}^2/\text{m}$
$\phi M_s = \phi A_s f_y j_u d$	$\phi M_s = \phi A_s f_y j_u d$
$= 0.9 \times 747 \times 500 \times 0.95 \times 100$	$=$ " " $\times 150$
$= 31.93 \text{kNm/m}$	$= 47.90 \text{kNm/m}$

NB! Minor undercapacity in Foundation for closing moment case. (1.125 factor = 89%)

CAPACITY OF PANEL / FOUNDATION JOINT:-
CHECK TCM 12 INSERTS AS PER KOA DWG 52 SECTION 1/S1.

REFER ANCHOR WORKSHEET A1 GIVES

$\phi N_{ur} = 12.67 \text{kN}$
$N^* = 36 \text{kN}$] $N^*/\phi N_{ur} >> 1.$ NG

REFER SECOND CHECK BASED ON ALTERNATIVE ANALYSIS ASSUMING FULL DEPTH OF FOUNDATION ROCK BEING EFFECTIVE IN TERMS OF SUPPORTING BOTTOM TCM 12 ANCHOR

GIVEN THIS, NO NEED TO CHECK SHEAR IN PANEL BELOW TOP ROW OF TCMS.

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CLIENT REF. NO:
BY: B J BROWN CHECKED:
JOB No/PAGE: 49158/1/03
DATE: 15/5/17

ALTERNATIVE ANALYSIS BASED ON FULL DEPTH OF FOUNDATION EDGE BEAM BEING EFFECTIVE IN PROVIDING ANCHORAGE FOR BOTTOM TCM ROW.

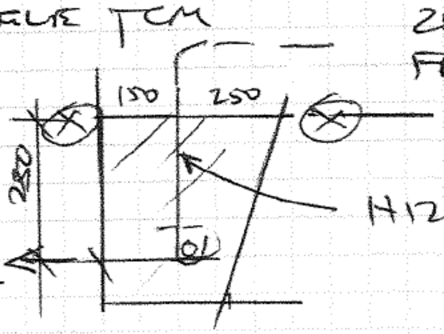
RECALC INSERT TCM DEMAND, BASED ON $S = 400mm$

$$T_0 = \frac{M_{os}}{S} = \frac{35.92 \text{ kNm/m}}{0.40 \text{ m}} = 89.8 \text{ kN/m}$$ CLOSING OR OPENING

$$= 89.8 \text{ kN/m} \times 0.15 \text{m c/c}$$

$$= 13.47 \text{ kN/Anchor}$$

DOWNSTAND RESISTANCE: FOR SINGLE TCM



$$M^*_{xx} = T \times d$$

$$= 13.47 \text{ kN/anchor} \times 0.250$$

$$= 3.36 \text{ kNm/bar}$$

$$\phi M_{sc} = 0.9 \times 112 \times 500 \times 0.95 \times 150$$

$$= 7.2 \text{ kNm/bar}$$

$$> M^*_{sc} \quad \text{OK}$$

$$\phi V_s = 0.75 \times 150 \times 150 \times 0.85 \text{ MPa} \; [0.17\sqrt{f'c} = 0.85 \text{ MPa}]$$

$$= 14.3 \text{ kN/bar}$$

$$> V^* \quad \text{OKAY JUST}$$

CHECK TCM 12 TENSILE CAPACITY, GIVEN REDUCED EDGE DISTANCE TO BOTTOM OF 150 PANEL

REFER 'REID ANCHORAGE DESIGN WORKSHEET'

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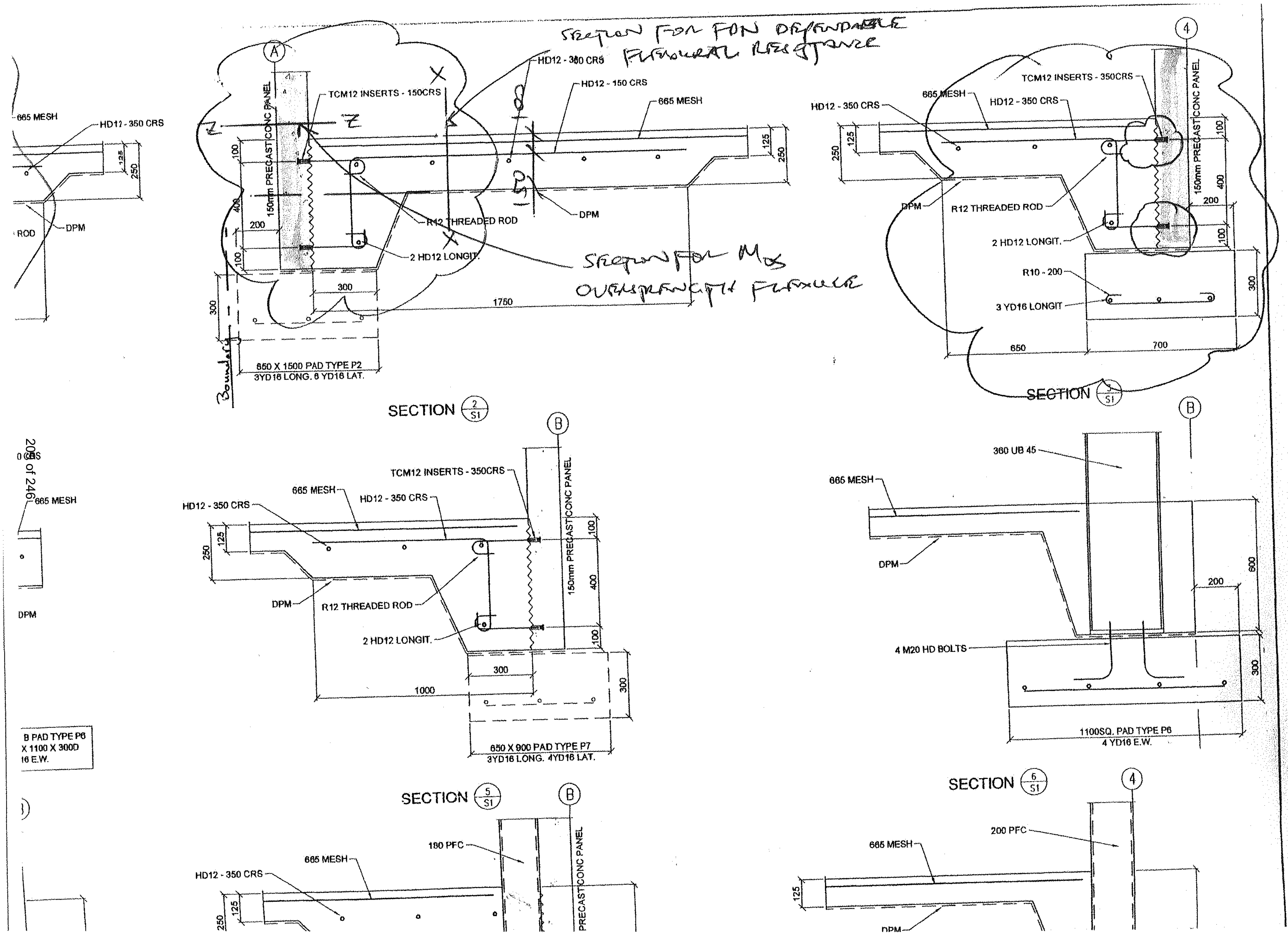
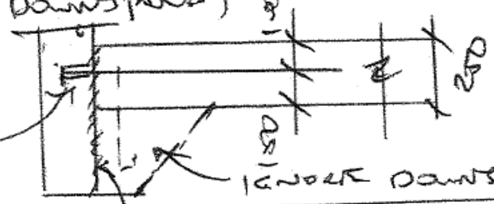

SECTION FOR FDN DEPENDABLE FLEXURAL RESISTANCE
SECTION FOR Mo OVERSTRENGTH FLEXURE
Boundary
HD12 - 380 CRS
HD12 - 150 CRS
665 MESH
TCM12 INSERTS - 150CRS
150mm PRECAST CONC PANEL
R12 THREADED ROD
2 HD12 LONGIT.
DPM
1750
300
650 X 1500 PAD TYPE P2
3YD16 LONG. 6 YD16 LAT.
SECTION 2/S1
TCM12 INSERTS - 350CRS
HD12 - 350 CRS
R10 - 200
3 YD16 LONGIT
650
700
SECTION 3/S1
1000
650 X 900 PAD TYPE P7
3YD16 LONG. 4YD16 LAT.
SECTION 5/S1
360 UB 45
4 M20 HD BOLTS
1100SQ. PAD TYPE P6
4 YD16 E.W.
SECTION 6/S1
180 PFC
200 PFC
B PAD TYPE P6
X 1100 X 300D
16 E.W.

ANCHORING DESIGN WORKSHEET

Project	EARNE AUSTRATON
Design	408 Queen St
Location	GRID A/1-4
Project ID	KOA 102610
Design by	BJTB.
Date	16/8/17
Checked	

Sketch DWG 102610/S2 SECTION 2/S1 BASED ON 250 SLAB (NO DOWNSTAND)



Inputs

DEMAND SEISMATRY FOR PANEL M_{OS} CAPACITY CASE

N* & V* are the **per anchor** load demand.
Check both external and internal anchors for suitability.

Tensile design action effect	N*	36	kN
Shear design action effect	V*	N/A	kN
Substrate thickness	b_m	150	mm
Concrete compressive strength	f'_c	40	MPa

STEP 1 SELECT ANCHOR TO BE EVALUATED

Table 1a: Find the interaction of N* and V* values

Anchor Type: TCM 12 with WALLPLATE

Table 1a: Absolute minima, edge distance and half of anchor spacing.

e_m: 36mm

Check for compliance with absolute minima — tick ✓

Checkpoint 1:

Anchor size selected? — tick ✓

Comply with absolute minima? — tick ✓

STEP 2 VERIFY CONCRETE TENSILE CAPACITY

Table 2a: Concrete tensile capacity ϕN_{uc}: 14.9 kN

Table 2b: Concrete compressive strength effect > 32 MPa ψ_{nc}: 1.26

Table 2c and /or 2d: Edge distance and anchor spacing reduction factors

Factor	Type	Table	Value
ϕx_1	>200	2C	1.0
ϕx_2	150	2D	0.9
ϕy_1	–	–	–
ϕy_2	100	–	1.0

Step 2a: Cracked concrete reduction factor ψ_{CR}: 0.75

Checkpoint 2:
Calculate: $\phi N_{URC} = \phi N_{uc} * \psi_{nc} * \phi x_1 * \phi x_2 * \phi y_1 * \phi y_2 * \psi_{CR}$ = 12.67 kN

STEP 3 VERIFY STEEL TENSILE CAPACITY

Table 3a: Anchor steel capacity ϕN_{US}: 21.2 kN

Table 3b: Bolt steel capacity ϕN_{TF}: 56 kN

ϕ 2065

Checkpoint 3:
ϕN_{UR} = minimum of $\phi N_{URC}, \phi N_{US}, \phi N_{TF}$ = 12.67 kN

$N^* / \phi N_{UR} \leq 1.0$? 36 / 12.07 ≥ 3.0 — tick ✗

If not satisfied, return to step 1.

Tensile Design Completed

STEP 4 VERIFY CONCRETE SHEAR CAPACITY

NO CONCURRENT SHEAR CHECK UNDERTAKEN — REFER ALTERNATIVE DESIGN CASE CHECK.

Table 4a:	Concrete shear capacity	ϕV_B		kN
Table 4b:	Concrete compressive strength effect	ψ_{vc}		
Table 4c:	Anchor group factor	ψ_5		
Table 4d:	Edge distance factor	ψ_{6A}		
		ψ_{6B}		
Table 4e	Concrete crack failure	ψ_7		
Table 4f	Projected failure area when not limited by edge distance	A_{vc}		mm²

Step 4g Projected concrete failure area of anchor A_v

Edge distance / Half of anchor spacing, mm	Original value, A	Table 4g. 1.5c, B	Calc value, min (A,B)
c_{2a}			
c_{2b}			
b_m			
			$A_v = (c_{2a} + c_{2b}) * b_m$

Checkpoint 4: Calculate
$\phi V_{CB} = \phi V_B * \psi_{vc} * \psi_5 * \psi_{6A} * \psi_{6B} * \psi_7 * A_v / A_{vc}$ = ___ kN

STEP 5 VERIFY STEEL SHEAR STRENGTH ϕV_s

Table 5a: Steel shear capacity ϕV_{s1} ___ kN

Table 5b: Bolt shear capacity ϕV_{s2} ___ kN

STEP 6 VERIFY CONCRETE PRY-OUT STRENGTH IN SHEAR ϕV_{cp} (PER ANCHOR)

Table 6a: Pry-out co-efficient k_{cp} ___

Checkpoint 5:

Calculate $\phi V_{cp} = 0.75/0.6 * k_{cp} * \phi N_{URC} * \psi_{vc}$ ___ kN

Checkpoint 6:

ϕV_{UR} = Minimum of $\phi V_{CB}, \phi V_{S1}, \phi V_{S2}, \phi V_{CP}$ = ___ kN

$V^* / \phi V_{ur} \leq 1.0$? ___ / ___ = ___

STEP 7 COMBINED LOADING & SPECIFICATION

Checkpoint 7:
$N_* / \phi N_{UR} + V^* / \phi V_{UR} \leq 1.2$?
___ / ___ + ___ / ___ = ___ tick

If not satisfied, return to step 1.

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SPECIFY: ______

CLIENT: Iframe / Masterton Buildings

JOB: 408 Queen St

CLIENT REF. NO:

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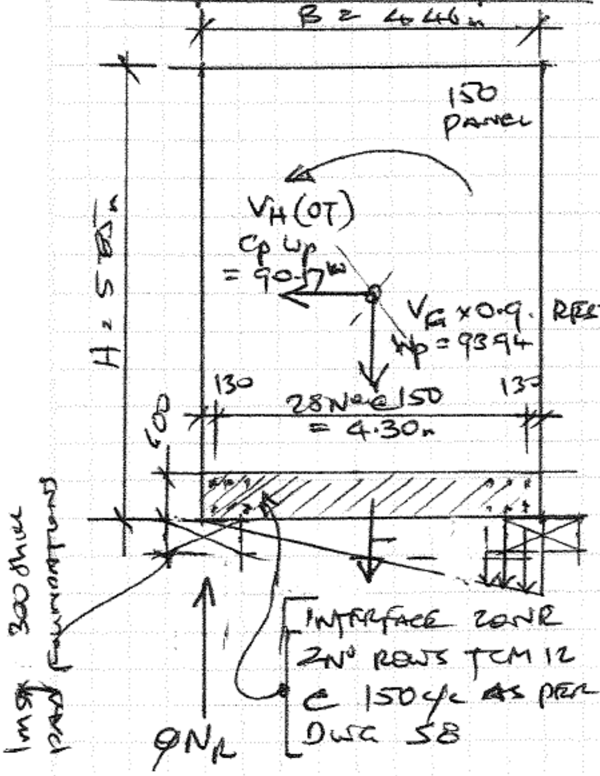
JOB No/PAGE: 49158/1/04

DATE: 15/5/17

GEOMETRY FOR WALL PANEL / FOUNDATION JUNCTION WHEN CHECKING OUT OF PLANE OVERTURNING RESISTANCE:



REACTION POINT ON 1000 SQ FOUNDATION PADS.

PANEL GEOMETRY: – 150 PANEL

GRAVITY LOADS

$q = 0.15 \times 2.4 kN/m^2 = 3.6 kN/m^2$

$A_p = B \times H = 4.46 \times 5.85m = 26.09m^2$

$W_p = q A_p = 26.09 m^2 \times 3.6 kN/m^2$

$= 93.94 kN$

SEISMIC OVERTURN LOADS / MOMENTS - FOR NZS 4203:1992 Cl. – REFER APPENDIX 'C'

$C_h(T_1) = C_h(\mu = 1, S_p = 1) = 0.959$

$V_H(OT) = C_h(T) W_p$

$= 0.959 \times 93.94 kN$

$= 90.17 kN \leftarrow$

$M_{O/T} = V_H(OT) \times H_g$

$= 90.17 kN \times \frac{5.850}{2}$

$= 263.7 kNm$

GRAVITY RESTORING LOADS / MOMENTS (DOWNRATED BY FACTOR @ 0.9.)

$M_{REST} = 0.9 \times 93.94 kN \times \frac{4.46}{2}$

$= 188.5 kNm$

STABILIZING DEMAND ON TCMS ACROSS INTERFACE ZONE

$M_{STAB} = M_{O/T} - M_{REST}$

$= 263.7 - 188.5$

$= 75.2 kNm$

$V_{STAB} = M_{STAB} \times \frac{2}{B}$

$= 75.2 kNm \times \frac{2}{4.46m}$

$= 33.7 kN$

V_{STAB} IS PROVIDED VIA SHEAR ACROSS 28No TCM PAIRS ACROSS B = 4.46m OF WALL PANEL

$(V_{TCM})^* = \frac{33.7 kN}{\text{AVERAGE } 28No \times 2 \text{ ROWS}} = 0.60 kN/TCM$ AVERAGE

THE ANNUAL ALLOCATION REQUIREMENT

$(V_{TCM})^*_{PEAK} = 2 \times (V_{TCM})$ AVERAGE

$= 1.20 kN/TCM.$

ADD TO TCM V^* THE RESISTANCE TO $V_H(OT) = 90.17 kN$ FROM HORIZONTAL BASE SHEAR

$(V_{TCM})_H = \frac{V_H}{2N}$

$= \frac{90.17 kN}{28No \times 2}$

$= 1.61 kN/TCM$

RESULTANT DEMAND ON TCM. SUBJECT TO:

(I) AVERAGE DEMAND

$V_{TCM} = \sqrt{(0.60)^2 + (1.61 kN)^2}$ (0.36, 2.59)

$= 1.71 kN/TCM$

(II) PEAK DEMAND: –

$V_{TCM} = \sqrt{(1.20)^2 + (1.61)^2}$ (4.03)

$= 2.01 kN/TCM$

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ANCHORING DESIGN WORKSHEET

Project	IPFRE / MASTERTON
Design	[redacted]
Location	GRID A / 1-4
Project ID	KOA 102610
Design by	BJB
Date	16/8/17
Checked	

Sketch: 4915B/1 FLEXURALANALYSIS AS PER CALCULATIONS SHEET C3

Inputs

N* & V* are the **per anchor** load demand.
Check both external and internal anchors for suitability.

Tensile design action effect	N*	13.5	kN
Shear design action effect	V*	2.0	kN
Substrate thickness	b_m	150	mm
Concrete compressive strength	f'_c	40	MPa

STEP 1 SELECT ANCHOR TO BE EVALUATED

Table 1a: Find the interaction of N* and V* values

Anchor Type: TCM12 @ 150 WITH NAILPLATE
EMBEDMENT = 55 + 8 = 63

Table 1a: Absolute minima, edge distance and half of anchor spacing.

e_m: 36 mm

Check for compliance with absolute minima — tick ✓

Checkpoint 1:

Anchor size selected? — tick ✓

Comply with absolute minima? — tick ✓

STEP 2 VERIFY CONCRETE TENSILE CAPACITY

Table 2a: Concrete tensile capacity	ϕN_{uc}	14.9	kN
Table 2b: Concrete compressive strength effect	ψ_{nc}	1.26	

Table 2c and /or 2d: Edge distance and anchor spacing reduction factors

Factor	Value	Table	Value
ϕx_1	>200	2c	1.0
ϕx_2	>150	2d	0.9
ϕy_1	–	–	–
ϕy_2	–	–	–

Step 2a: Cracked concrete reduction factor ψ_{CR} 0.75

Checkpoint 2:
Calculate: $\phi N_{URC} = \phi N_{UC} * \psi_{nc} * \phi x_1 * \phi x_2 * \phi y_1 * \phi y_2 * \psi_{CR}$ = 12.67 kN

STEP 3 VERIFY STEEL TENSILE CAPACITY

Table 3a: Anchor steel capacity	ϕN_{US}	21.2	kN
Table 3b: Bolt steel capacity	ϕN_{TF}	56	kN

Checkpoint 3:
ϕN_{UR} = minimum of $\phi N_{URC}, \phi N_{US}, \phi N_{TF}$ = 12.67 kN

$N^* / \phi N_{UR} \leq 1.0$? 12.50 / 12.67 = 1.065 — tick ☒
NON - COMPLIANT

If not satisfied, return to step 1. — Tensile Design Completed

STEP 4 VERIFY CONCRETE SHEAR CAPACITY

Table 4a:	Concrete shear capacity	ϕV_B	55.3	kN
Table 4b:	Concrete compressive strength effect	ψ_{VC}	1.26	
Table 4c:	Anchor group factor	ψ_5	1.0	
Table 4d:	Edge distance factor	ψ_{6A}	1.0	
		ψ_{6B}	1.0	
Table 4e	Concrete crack failure	ψ_7	1.0	
Table 4f	Projected failure area when not limited by edge distance (CRACKED / NO REO ACROSS)	A_{vo}	72.0	$\times 10^4$

Step 4g Projected concrete failure area of anchor A_v

Edge distance / Half of anchor spacing, mm	Original value, A	Table 4g, 1.5c, B	Calc value, min (A,B)
c_{2a}	75	MIN. c_1 = 100	75
c_{2b}	75	$1.5c_1$ = 150	75
b_m	150		150
		$A_v = (c_{2a} + c_{2b})*b_m$	2.25 $\times 10^4$

S = 150 TYPICAL

Checkpoint 4: Calculate
$\phi V_{CB} = \phi V_B * \psi_{VC} * \psi_5 * \psi_{6A} * \psi_{6B} * \psi_7 * A_v / A_{vo}$ = 2.18 kN
69.68 kN
A_v / A_{vo} = 2.25 / 72.0 = 0.0312

STEP 5 VERIFY STEEL SHEAR STRENGTH ϕV_s

Table 5a: Steel shear capacity ϕ = 0.75	ϕV_{s1}	27.8	kN
Table 5b: Bolt shear capacity G8.8	ϕV_{s2}	29.3	kN

STEP 6 VERIFY CONCRETE PRY-OUT STRENGTH IN SHEAR ϕV_{cp} (PER ANCHOR)

Table 6a: Pry-out co-efficient k_{cp} 1.0
h = 55 + 8 = 63

Checkpoint 5:

Calculate $\phi V_{CP} = 0.75/0.6 * k_{cp} * \phi N_{URC} * \psi_{VC}$ = 19.9 kN

Checkpoint 6: $\left[\frac{0.75}{0.6 \times 1.0}\right] \times 12.67 \times 1.26$

ϕV_{UR} = Minimum of $\phi V_{CB}, \phi V_{S1}, \phi V_{S2}, \phi V_{CP}$ = 2.25 kN

$V^* / \phi V_{ur} \leq 1.0$? 2.0 / 2.18 = 0.92 ✓ COMPLIANT

STEP 7 COMBINED LOADING & SPECIFICATION

Checkpoint 7:
$N_* / \phi N_{UR} + V^* / \phi V_{UR} \leq 1.2$?
13.5/12.67 + 2.0 / 2.18 = 1.99(2) — tick ☒
1.06(6) 0.92(6)
NOT SATISFIED >> 1.2

If not satisfied, return to step 1.

SPECIFY: SEPARATE 45° Rd DIRECTION CHECK

$0.71 \times \left[\frac{13.5}{12.67} + \frac{2.0}{2.18}\right] = 0.71 \times 1.992 = 1.414 > 1.2$

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CLIENT: FENZ / MASTERTON BUILDINGS

JOB: 408 Queen St

CLIENT REF. NO:

BY: B-J. BROWN CHECKED:

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JOB No/PAGE: 49158/1/05

DATE: 15/5/17

SENSITIVITY CHECK — WHAT IF RAKER LEVEL PROPPING IS EFFECTIVE IN REDUCING THE PANEL / FOUNDATION BASE MOMENT DEMAND.

1. IF ALTERNATIVE (2ND FCM ROWS) ARE EFFECTIVE, THEN FCM DEMAND IS ~

$$T = 13.5\,kN \times 0.376 = 5.08\,kN.$$

2. QUERY CHECKPOINT #3: INTERACTION

$$5.08\,kN / 12.67\,kN = 0.40 < 1.0 \quad \text{OKAY} \checkmark$$

3. QUERY CHECKPOINT #6:

SAY 75% OF SHEAR DEMAND ON INTERFACE / BAR CONNECTION, WHEN CHECKING INTERACTION EQUATION.

$$1.50 / 2.18 = 0.69 < 1.0 \quad \text{OKAY} \checkmark$$

4. INTERACTION CURVE / CHECKPOINT #7.

$$N^* / \phi N_t + V^* / \phi V_u = 0.40 + 0.69 = 1.09 < 1.2 \quad \text{SO COMPLIES} \checkmark$$

HENCE IF PORTAL RAKER BEAM (TRANSVERSE) PROPPING IS EFFECTIVE THEN THE DEMAND REDUCES TO AROUND 40% OF THE PURE CANTILEVER DEMAND AND COMPLIANCE WITH INTERACTION EQUATION IS ACHIEVED.

Appendix C5.8

Representative calculations for the wall panel/foundation "TCM" connection

49158/4 - 57-65 (Lot 9) Dixon St

REVIEW OF KOA CALCULATIONS SECTION C COVERING THE WALL PANEL/FOUNDATION CONNECTION ON 57-65 (Lot 9) Dixon St (ref 49158/4 in Appendix B4)

In this section I consider the source of the under-capacity in the panel/foundation connection referred to at item 6.2 in the review assessment for the building given in Appendix B4.

In my opinion, the designer has assumed that the overturning moment in the slab can be resisted by a force couple comprising the cast-in insert and a reaction point at the base of the panel some 300 mm below (refer KOA calculations at pages C3 (sketch), C6 (ht = 300 mm), and C7 (M insert)). This assumption would be valid if the edge thickening of the slab was 400 mm deep (reference Section 1/S01 on drawing S12) for the full length of the wall panel, but does not, in this case, because this detail only occurs at the pile support. In reality, the base of the panel is unrestrained as per Section 6/S01 on drawings S13 and the 300 mm lever arm cannot apply.

By my assessment, the actual level arm to the TCM connection can only be around 100 mm, so that the resisting moment available can only be around 8 kN-m/m or around 30% of what is required.

As noted in Appendix C5 of this report, the foundation/wall panel connection incorporating the TCM anchorage must be designed at least to resist the overstrength moment in the wall panel, which is demonstrably not the situation with the details proposed for use in this building.

As part of my review, I have prepared some preliminary design calculations for the TCM inserts intended to reinforce the wall panel/foundation junction for the resistance of both out-of-plane face/bending loads and in-plane (shear) loads in a typical panel within this building.

These calculations suggest that these TCM connections in this particular building are significantly under-designed - ie say 20% of a fully compliant requirement - for the critical severe seismic (ULS) design case required by the relevant loading standard - refer Appendix C5 calculations ref 49158/4/01-05.

CLIENT FPRNZ / MASTERTON BUILDINGS

JOB 57-65 (Lot 9) Dixon St REF 110 286

CLIENT REF. NO:

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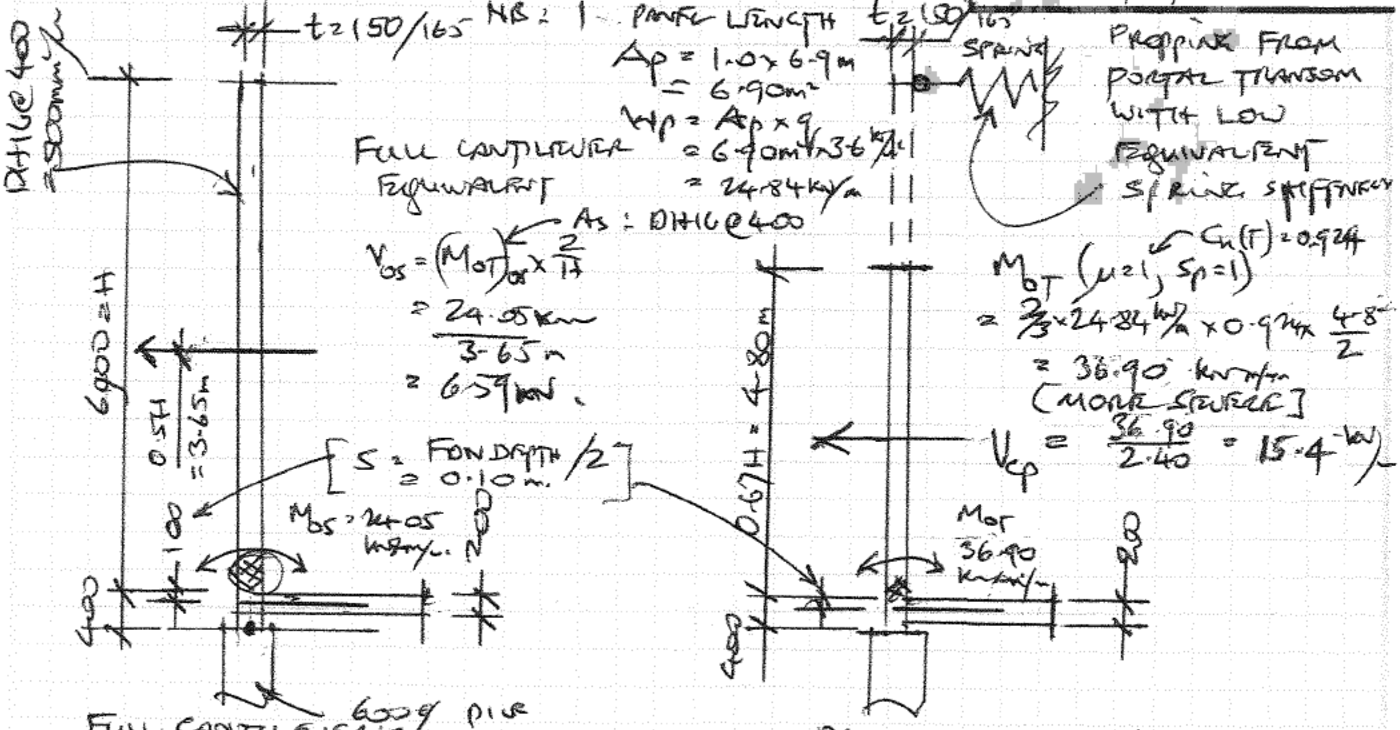
JOB No/PAGE 4958/4/01

DATE 16/5/17

1. GEOMETRY FOR WALL PANEL / FOUNDATION JUNCTION FOR CHECKING OF FACE LOAD MOMENT RESISTANCE



FULL CANTILEVER:-

STABILITY UNDER M_{os} LOAD CASE

$(M_{OT})_{os} = 1.35 A_s f_y jd$

$= 1.35 \times 500mm \times 500MPa \times 0.95 \times 75$

$= 24.05 kNm$ ✱

NB. Dependable $= \frac{0.9}{1.35} \times 24.05$

$= 16.03 kNm/m$

FOUNDATION DEPTH

D = 200mm

S = 100mm (OPENING / CLOSING)

PANEL THICKNESS 165mm

TCM INSERT LOAD DEMAND :-

$T_0 = \frac{M_{os}}{S} = \frac{24.05}{0.10m}$ $\left[\frac{26.46}{0.1}\right]$

= 240.5 kN/m [ ] (OPENING/CLOSING)

NB: EQUIVALENT TO 96.2 kN/TCM @400c/c

FOUNDATION RESISTANCE TO PANEL OVERTURNING, ASSUMING BEARING WIDTH OK

DH16 @ 400 IN 200 THICK FDN.

(OPENING / CLOSING MOMENT)

d = 100

As = 500mm²/m

$\phi M_s = \phi A_s f_y jd$

$= 0.9 \times 500 m^2 \times 500MPa \times 0.95 \times 100$

$= 21.38 kN\text{-}m/m < M_{os} = 24.05 kNm/m$ (88.8%)

PROPPED CANTILEVER STABILITY UNDER M (μ=1, Sp=1)

NZS1170.5 PART 5 (GLOBAL) OR PART 8 (PARTS)

DERIVES $C_H(T)$ = 0.924 (SEE NEXT PAGE) WHICH GEN GENERATES $M_{OT} > (M_{OT})_{os}$ SO DESIGN CASE WILL BE THAT DERIVED FROM FLEXURAL YIELD YIELD IN VERTICAL REINF

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CLIENT: [illegible] / MASTERTON BUILDINGS

JOB: 57-65 (Lot 9) Dixon St REF

CLIENT REF. NO:

BY: B J BROWN CHECKED:

JOB No/PAGE: 49158/4/02

DATE: 16/5/17

NB: CORRESPONDING ELASTIC RESPONSE DEMAND DERIVED AS:-

(i) NZS 1170.5 ($T = 0.35s$) RESPONSE ($\mu = 1$, $S_p = 1.0$)

$C_H(T) = 0.840$

(ii) SCALE BY 10% FOR 100% X, 30% Y DEMAND

$\rightarrow C_H(T) = 1.10 \times 0.840 = 0.924$.

CAPACITY OF PANEL/FOUNDATION JOINT :-

CHECK TCM 16 INSERTS AS PER KOA DWG S12 / DETAIL 5/S01, AND 6/S01.

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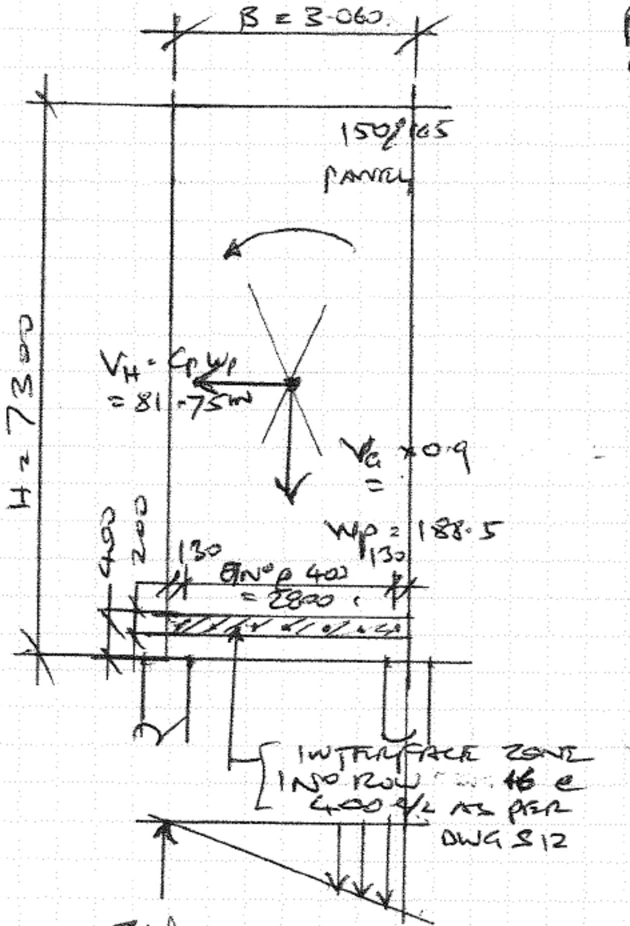
CLIENT: IFENZ / MASTERTON BUILDINGS
JOB: 57-65 (Lot 9) Dixon St   REF 1.10 286
CLIENT REF. NO:
BY: B. J. BROWN   CHECKED:

JOB No/PAGE: 49158/4/03
DATE: 15/5/17

GEOMETRY FOR WALL PANEL / FOUNDATION JUNCTION WHEN CHECKING INPLANE OVERTURNING RESISTANCE.



PANEL GEOMETRY – 150/165 PANEL
GRAVITY LOADS:–

$q = 0.165 \times 24\,kN/m^2 = 3.96\,kN/m^2$

$A_p = BH = 3.060m \times 7.30m = 22.34$

$W_p = A_p q = 22.34\,m^2 \times 3.96\,kN/m^2$
$= 88.47\,kN.$

SEISMIC OVERTURN LOADS / MOMENTS FOR NZS 1170.5 cl. – REFER APPENDIX 'C' SUMMARY FOR COEFFICIENT $C_H(T)$ DERIVATION

$C_H(T_1) = C_h(\mu=1, S_p=1) = 0.840$

SCALED (100% X, 30% Y) → 1.10 FACTOR
$= 0.924$

$V_H(OT) = C_h(T) \times W_p$
$= 0.924 \times 88.47\,kN$
$= 81.75\,kN$ →

$M_{OT} = V_H(OT) \times H_y$
$= 81.75\,kN \times \frac{7.30}{2}$
$= 298.4\,kNm$

GRAVITY RESTORING LOADS / MOMENTS (DOWNRATED BY FACTOR @ 0.9)

$M_{REST.} = 0.9 \times 88.47\,kN \times \frac{3.060}{2}$
$= 121.82\,kNm$

STABILIZING DEMAND ON TCMS ACROSS INTERFACE ZONE

$M_{STAB} = M_{OT} - M_{REST}$ (kNm)
$= 298.4 - 121.8$ (kNm)
$= 176.6\,kNm.$

$V_{STAB} = M_{STAB} \times \frac{2}{B}$
$= 176.6\,kNm \times \frac{2}{3.06m}$
$= 111.4\,kN.$

ADD TO $(V_{(TCM)})^*$ THE IN PLANE RESISTANCE TO $V_H = 81.75\,kN$ FROM HORIZONTAL BASE SHEAR

$(V_{TCM})_H = \frac{V_H}{N}$
$= \frac{81.75\,kN}{8N°}$
$= 10.22\,kN/TCM.$

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RESULTANT DEMAND ON TCM, WHEN SUBJECT TO:-

(1) AVERAGE SHEAR DEMAND.

$$V_{TCM} = \sqrt{(10.22)^2 + (13.93)^2}$$
$$= 17.3 \text{ kN}$$

(2) PEAK SHEAR DEMAND

$$V_{TCM} = \sqrt{(10.22)^2 + (27.86)^2}$$
$$= 29.67 \text{ kN}.$$

V_{SPAN} IS PROVIDED VIA SHEAR ACROSS 8No TCMs IN SINGLE ROW ACROSS B = 3060 m OF WALL PANEL.

$$(V_{TCM})^*_{AVE} = \frac{111.4 \text{ kN}}{8N^o} = 13.93 \text{ kN/TCM}$$

TRIANGULAR DISTRIBUTION OF SHEAR LOADS (SIMILAR TO BOLT GROUP ON GUSSET) RESULTS IN:

$$(V_{TCM})^*_{PEAK} = 2 \times (V_{TCM})^*_{AVERAGE}$$
$$= 27.86 \text{ kN}.$$

REVIEW TENSILE AND SHEAR CAPACITY OF TCM ARRANGEMENT USING REID 'ANCHORING DESIGN SHEET' AS ATTACHED, TO COVER SEPARATE SCENARIOS

1) FACE LOAD ON PANELS – TENSION ONLY ON TCM. (T)

(2) INPLANE LOAD ON PANELS. – SHEAR ONLY ON TCM. (V_H, V_{SPAN})

(3) COMBINED TENSION/SHEAR ON TCM, VIA. INTERACTION FORMULAE FOR APPLICABLE LOAD CASES. EG. 100% X + 30% Y

NOTE THAT FOR RESISTANCE TO CAPACITY DESIGN ACTIONS, EVALUATION OF COMBINED EFFECTS MUST INCORPORATE AN ADDITIONAL DE-RATING OF CAPACITY BY FACTOR 0.75, AS PER NZS 3101: 2006 Cl.17.6.4 ('FIXINGS DESIGNED TO REMAIN ELASTIC') REF EQTNS C17.6, C17.7 AND C17.8 IN COMMENTARY.

ANCHORING DESIGN WORKSHEET

Project	IDENZE / MASTERTON
Design	57-65 (Lot 9) Dixon St
Location	GRID. A/1-7
Project ID	KOA 110286
Design by	BJB
Date	16/5/17
Checked	

Sketch: DWG 110286/S12 SECTION 5/S01, AND 6/S01'. BASED ON 200 THICK FOUNDATION SLAB (SINGLE ROW TEMS WITH NO DOWNSTAND EDGE TO SUPPORT SECOND ROW. OF TEMS

CASE 1: TENSION ON TEMS FROM FACE LOADS ON PANELS

Inputs

N^* & V^* are the **per anchor** load demand.
Check both external and internal anchors for suitability.

Tensile design action effect	N^*	96.2	kN
Shear design action effect	V^*	29.7	kN
Substrate thickness	b_m	165	mm
Concrete compressive strength	f'_c	40	MPa

STEP 1 SELECT ANCHOR TO BE EVALUATED

Table 1a: Find the interaction of N^* and V^* values

Anchor Type: RB16TI @ 400 WITH NAILPLATE EMBEDMENT 118 + 8 = 126.

Table 1a: Absolute minima, edge distance and half of anchor spacing.

e_m: 48mm

Check for compliance with absolute minima — tick ✓

Checkpoint 1:

Anchor size selected? — tick ✓

Comply with absolute minima? — tick ✓

STEP 2 VERIFY CONCRETE TENSILE CAPACITY

Table 2a: Concrete tensile capacity ϕN_{uc} 29.5 kN

Table 2b: Concrete compressive strength effect Ψ_{nc} 1.26

Table 2c and /or 2d: Edge distance and anchor spacing reduction factors

Factor	Type	Dist	Value
ϕx_1	EDGE	2C	115/1.00
ϕx_2	SPACER	2D	400/1.00
ϕy_1	EDGE	2C	200/1.00
ϕy_2	EDGE	2C	200/1.00

Step 2a: Cracked concrete reduction factor Ψ_{CR} 0.75

Checkpoint 2: 29.5 kN, 1.0, 0.75

Calculate: $\phi N_{URC} = \phi N_{uc} * \Psi_{nc} * \phi x_1 * \phi x_2 * \phi y_1 * \phi y_2 * \Psi_{CR}$ = 22.1 kN

STEP 3 VERIFY STEEL TENSILE CAPACITY

Table 3a: Anchor steel capacity ϕ = 0.65 ϕN_{us} 33.2 kN

Table 3b: Bolt steel capacity ϕ = 0.80 ϕN_{TF} 104.2 kN

Checkpoint 3: 22.1, 33.2, 104.2

ϕN_{UR} = minimum of $\phi N_{URC}, \phi N_{US}, \phi N_{TF}$ = 22.1 kN

$N^* / \phi N_{UR} \le 1.0$? 96.2 / 22.1 = 4.35 > 1 — tick ☒

If not satisfied, return to step 1.

Tensile Design Completed

STEP 4 VERIFY CONCRETE SHEAR CAPACITY

Table 4a:	Concrete shear capacity	ϕV_B	64.0	kN
Table 4b:	Concrete compressive strength effect	Ψ_{vc}	1.26	
Table 4c:	Anchor group factor	Ψ_5	1.00	
Table 4d:	Edge distance factor	Ψ_{6A}	1.00	
		Ψ_{6B}	1.00	
Table 4e	Concrete crack failure (CRACKED / NO REO ACROSS)	Ψ_7	1.00	
Table 4f	Projected failure area when not limited by edge distance $A_{vo} = 4.5c_1^2$	A_{vo}	40.5	mm² × 10⁴

Step 4g Projected concrete failure area of anchor A_v

Edge distance / Half of anchor spacing, mm	Original value, A	Table 4g, 1.5c, B	Calc value, min (A,B)
c_{2a}	200	MIN c_1 = 300	200
c_{2b}	200	1.5c_1 = 450	200
b_m	165		165
		$A_v = (c_{2a} + c_{2b}) * b_m$	6.60 × 10⁴

S = 400 TYPICAL

Checkpoint 4: Calculate 80.64 kN

$\phi V_{CB} = \phi V_B * \Psi_{vc} * \Psi_5 * \Psi_{6A} * \Psi_{6B} * \Psi_7 * A_v / A_{vo}$ = 13.14 kN

A_v / A_{vo} = 6.60 / 40.5 = 0.1629

STEP 5 VERIFY STEEL SHEAR STRENGTH ϕV_s

Table 5a: Steel shear capacity ϕ = 0.75 ϕV_{s1} 23.0 kN

Table 5b: Bolt shear capacity G8.8 ϕV_{s2} 52.1 kN

STEP 6 VERIFY CONCRETE PRY-OUT STRENGTH IN SHEAR ϕV_{cp} (PER ANCHOR)

Table 6a: Pry-out co-efficient k_{cp} 2.0

h = 75+8 = 83 > 65

Checkpoint 5:

Calculate $\phi V_{CP} = \{0.75/0.6 * k_{cp}\} * \phi N_{URC} * \Psi_{VC}$ = 17.4 kN

Checkpoint 6: $\left[\frac{0.75}{0.6} \times 2.0\right] \times 22.1 \times 1.26$

ϕV_{UR} = Minimum of $\phi V_{CB}, \phi V_{S1}, \phi V_{S2}, \phi V_{CP}$ = 13.14 kN

$V^* / \phi V_{ur} \le 1.0$? 29.7 / 13.14 = 2.26 > 1.0 : PEAK

17.3 / 13.14 = 1.32 > 1.0 : AVERAGE

STEP 7 COMBINED LOADING & SPECIFICATION

Checkpoint 7: 4.35 + 2.22 = 6.57 > 1.2

$N_* / \phi N_{UR} + V^* / \phi V_{UR} \le 1.2$? 96.2/22.1 + 29.7/13.4 = 6.57 — tick ☒

NOT SATISFIED >> 1.2

If not satisfied, return to step 1.

SPECIFY: NON - COMPLIANT

reid™

Fraser Thomas

ENGINEERS • RESOURCE MANAGERS • SURVEYORS

30 April 2018 49158

Principal Advisor – Special Projects
IPENZ/Engineers New Zealand
PO Box 12 145
Thorndon
WELLINGTON 6144

Attention Investigator

Investigator

RE: STRUCTURAL COMPLIANCE REVIEW – SIX MASTERTON BUILDINGS
EXPERT ADVICE FOR IPENZ OWN MOTION INQUIRY
FURTHER ADVICE REQUEST FROM INVESTIGATING COMMITEE

1. I refer to your request by letter dated 11/4/18, where the Investigating Committee (IC) has sought additional advice on two points viz:-

   (1) General processes used to develop design documentation viz "plans and specifications" for buildings of the subject type at the time of consent, particularly insofar 'quality assurance', and 'CPEng sign out' was concerned, and

   [redacted] [redacted]

2. In responding, I will address item (1) (the substantive item) against the two question put by the IC regarding the use of design draftspersons on the generation of the "drawing development work", with the concept/detailed calculations being checked by a senior staff engineer.

   Q1(a): Whether the process described reflected reasonable practice at the time that each of the six buildings were consented, and

   Q1(b): If the process was reasonable, then was it appropriate that a PS1 sign off should have been given by the CPEng concerned, i.e. errors, and/or omissions within concept and/or detail being accepted, as the authors responsibility.

   These two aspects are addressed within items 3 and 4 below.

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3. Prior to responding to these, I have taken into account what I believe was normal practice within design offices over the period concerned.

   (a) I find this well described within IPENZ Practice Note #14 “Structural Engineering Design Office Practice” (V1 August 2009), although I believe it is useful to consider separately the pros/cons of the methodology reportedly described by Mr O’Connor for the type of buildings concerned.

   (b) To do this, I have prepared some background notes (refer Appendix B) which addresses in general terms the pros/cons of the processes reportedly used by Mr O’Connor (and others with his firm) before they provided the design compliance (PS1) sign off’s on the buildings concerned.

   (c) I have included my review comment on the six Masterton Buildings in item 4 following.

4. My comments on design/documentation standards achieved in particular buildings as signed off by Mr O’Connor (and/or others within his firm,) draws on Appendix B (Sections B1 through B6) of my previous report dated 31/10/17.

   NB: Consenting dates for each building included with the heading, as these dates are relevant in terms of the B1/VM1 (and other industry documents) in use at the time.

4.1 Appendix B1: Masterton/ 408 Queen St [3/2003]

   (a) I suspect the precast wall panel elevations shown on Dwg S2 were developed as a standard solution by a design drafting process.

   (b) I suspect the fail critical design concept viz site welded lintel panel, was a draftsman generated detail. CPEng/ Principal sign off was seriously flawed in terms of this detail.

4.2 [redacted]

   [redacted]

   [redacted]

4.3 Appendix B3: 57-65 Dixon St [12/2006]

   (a) As noted previously (ref Appendix B3 item 7.1) the structural concept in this building is reasonably well developed, and load paths logical.

   (b) My criticism of this design is mainly engineering related, and second order in seriousness (apart from slab/wall TCM joint connection).

   (c) I suspect the engineering input on this building was timely, and the drafting work reasonably well supervised.

4.4 Appendix B4: 57-65 (Lot 9) Dixon St [10/2010]

   (a) As noted previously (ref Appendix B4 Items 7.1(a) the structural concept for this building is reasonably well developed, and the load paths logical.

(b) Because this building was designed post B1/VM1 transition date viz 2009-2010, it required more focused engineering input in terms of e.g. its ability to withstand seismic overload.

(c) My criticisms of this building are mainly engineering related, e.g. wall panel, foundation slab TCM joint connection.

4.5 Appendix B5: 196-120 Queen St [7/2011]

(a) As noted previously (Appendix B5 item 6.1), the structural concept for the building is reasonably well developed, although some critical items with the lateral force resisting system could need further review.

(b) Because this building was designed post B1/VM1 transition viz 2009-2010 it required more focused engineering input in terms of e.g. its ability to withstand seismic overload.

(c) My criticism of this building relates primarily to structural engineering aspects, e.g. wall panel/ foundation slab joint connection (TCMS), and the sufficiency of the R24 brace connection.

(d) Notwithstanding the above, I suspect the engineering input on this building was timely, and the drafting work reasonably well supervised.

4.6 Appendix B6: Cnr Dixon and Church St [4/2014]

(a) As noted previously (Appendix B6 item 5.1, 5.2 and 6.1), the structural concept for this building is unusual with complex load paths throughout and not well developed, with a mix of stiff and flexible elements not well matched in many instances.

(b) Because this building was designed post B1/VM1 transition viz 2009-2010 it required more focused engineering input in terms of e.g. its ability to withstand seismic overload, particularly at serviceability limit state.

(c) I note that structural calculations and drawings are both dated 12/2013, suggesting drafting and structural verification were carried out together. My suspicion is that this building was conceptualised by a design draftsperson, with engineering input being overlaid in an ad hoc manner after the primary design decision had been made by the drafter.

(d) In general, I found this building to be the worst example within the set in terms of being a "poorly developed structural concept".

[redacted] [redacted]

[redacted] [redacted]

[redacted] [redacted]

[redacted] [redacted]

[redacted] [redacted]

Fraser Thomas

[redacted]

5.2 I trust the above is sufficient response to this question.

6. Please contact me if the above responses are insufficient, or further information is required.

Yours sincerely
FRASER THOMAS LTD

BARRY BROWN
Principal - Structural Engineering

Appendices

A IPENZ/ Julia Byers letter dated 11/4/18 (2 pages)
B B Brown/Notes on Documentation Process (2 pages)

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APPENDIX A:

REGISTRATION AUTHORITY
FOR CHARTERED PROFESSIONAL ENGINEERS

11 April 2018

Barry Brown
By email: [redacted]

Dear Barry

FURTHER ADVICE REQUEST FROM INVESTIGATING COMMITTEE

Thank you for meeting with the Investigating Committee on 1 March. This was a helpful meeting.

The Investigating Committee have now completed interviews with the engineers involved with the sign off of these buildings — Kevin O'Connor, [redacted]

To assist the Investigating Committee with their consideration of these cases, they would like some additional advice from you, in particular:

1. During the recent interviews Mr O'Connor advised that the following general process was followed in relation to quality assurance and sign out. He advised that this changed from about 2012, at which time the process was more rigorous and documented:
   - The concept design was carried out by a senior engineer, but not necessarily a CPEng engineer.
   - A draftsperson did all the drawing development work.
   - The drawings and calculations were checked by a senior engineer (not CPEng).
   - A CPEng engineer would carry out a high-level review of the designs and calculations and sign the PS1 on verbal assurance from the senior engineer that the designs and calculations had been checked.

   a. Please advise if the above process is consistent with reasonable practise at the time. Please provide your answer for each building, taking into account the year that they were designed.

   b. If you consider that this process was reasonable, please advise whether you consider it was reasonable and appropriate for each of the buildings to have been signed out, by way of PS1, at the time.

   For example, in the case of 196-120 Queen St was it reasonable for Mr O'Connor to have signed out the building by way of a PS1 in the circumstances, or do you consider that the issues identified with the design and building system were such that Mr O'Connor should have identified these during his high-level review prior to signing the PS1?

   Again, please provide your answer for each individual building.

[redacted]

If you have any questions about the above request please let me know by email:

[redacted]

It would be helpful to receive your advice by 30 April 2018.

Thank you for your assistance.

Yours sincerely

[redacted]

Principal Advisor — Special Projects

49158

IPENZ OWN MOTION INQUIRY – SIX MASTERTON BUILDING

APPENDIX B: Notes on Processes for Developing Design Documentation for Simple Standardised Commercial/Industrial Building

1. Firstly, as I have said previously, the buildings in question are of repeatable "Commodity" type, incorporating a simplified design approach using an assembly of what are essentially standardised structural components.

2. Secondly, notwithstanding their nature, these buildings need to fit into a regulatory approval system prescribed by the Building Act (BA) requiring the demonstration (by calculation etc) of structural performance under various compliance standards that, in reality have become more rigorous and demanding over time, with the B1/VMI transition occurring around 2009/2010 using the most severe of these.

3. My response in terms of the general process for the development of design documentation described by Mr O'Connor could work for a particular building provided that:-

   (a) The concept design was formulated and agreed at a high level early on, with a widely accepted understanding across the design office of what are 'go/ no go' concepts and details to be used.

   (b) There was regular Senior Engineer/CPEng Principal input along the way that ensured that unsatisfactory concepts, or details which appeared did not become locked in before the documentation was budget was spent, and therefore difficult to change.

4. In reality, 'drawing development' will always involve 'design concept development', and the drafting led drawing development can be fraught unless there is regular and proactive intervention by the Senior Engineer/CPEng Principal along the way.

5. In my experience, the BRANZ Tilt-up Guideline (published 2005) was important development for this type of building in that it introduced a number of 'qualified performance concepts' that needed conscious consideration by the designer. The adoption of these concepts would not (and probably did not) occur where the concept/detailed design development was left with the drawing generator without Senior Engineer/CPEng Principal input at the appropriate stage.

6. Typically, in my experience, a drafting generated design will focus on drawing up from a library of 'standard details' previously developed on other similar projects, and applying these to new situations which may or may not be appropriate in a particular circumstance. This is where the Senior Engineer/CPEng Principal proactive inputs are critical, in order to recognise that:-

   (a) Some details previously used might (for various reasons) no longer be appropriate.

   (b) New details and/or concepts need to be developed to satisfy new (or better qualified) performance concepts.

7. Given all the above, it will never be sufficient for the CPEng Principal to sign off a design based on a verbal assurance from a Senior Engineer (as opposed to a first-hand review of the design output viz drawings, and calculations by the CPEng/ Principal, notwithstanding how brief that review might be.

8. In considering all of the above, I have kept in the front of my mind the following reality:-

(a) Regardless of any high level protocols that a design office might develop, the day to day commercial pressures may well result in prescribed ‘checklist’ activities getting out of sequence, i.e. the ‘slippery slope’ route to the precipice of inadequately reviewed work will always be there.

(b) The acid test will be how the CPEng Principal addresses a substandard product which emerges at the end of the drawing/design production process, within the environment created by such pressures.

9. Referring to the ‘improved documentation’ and rigour that was reportedly introduced by Mr O’Connor around 2012, based on the sample of buildings that I have reviewed my conclusions are that I am not convinced that this improvement was effective. For example, the design which, in my view, incorporated the poorest structural concept and detailing from a structural engineering perspective was that within Cnr Dixon and Church St [4/2014] which came after that date.

10. In the case of the Masterton buildings it is not always clear at which stage (date) the engineering calculations were undertaken, as compacted to the production of the drawings. I note that whilst the structural drawings are dated, the Engineers calculations are often undated.

11. Summary/Conclusions

In the final event, if the output PS1 for design (and/or PS4 (for construction review)) signed off by the CEng/Principal is defective in terms of its statements (and irrespective of the reliance placed on verbal advice given by a Senior Engineers), the author must accept the responsibility arising from that. Put another way, the process that the CPEng/ Principal used in reaching his/her judgement regarding the sufficiency of building design documentation is irrelevant if the statement is defective.

APPENDIX B FOR DOCUMENTS B1 - B5

EXPERT ADVICE REPORT FROM STUART GEORGE (REDACTED)

BGT Structures

12362

11 August 2017

IPENZ
Level 3
50 Customhouse Quey
Wellington,

Attention: Investigator

Re: Masterton Inquiry
Initial Report

Dear Sir,

Further to your instructions this report sets out my advice following a review of documentation provided on the engineering design of six buildings located in Masterton. My advice extends to an opinion as to whether the designs met accepted standards and were consistent with what a reasonable engineer in the same circumstances would likely have done.

I have arranged the buildings in chronological order as follows, because acceptable standards change with time, and building codes and best practice are always evolving, which is relevant to the conclusion I make.

Building Reference Name	*Year*	*KOA Job Reference*	*Quick Description*
Cnr Dixon and Church	2014	113352	Single storey retail building, tilt panel and metal roof.
196-120 Queen St	2011	111012	Single storey retail building with mezzanine floor, tilt panel and metal roof.
57-65 (Lot 9) Dixon St	2010	110286	Single storey retail building with mezzanine floor, tilt panel and metal roof.
57-65 Dixon St	2006	106265	Single storey retail building, tilt panel and metal roof.
[redacted]	[redacted]	[redacted]	[redacted]
408 Queen St [redacted]	2003	102610	Single storey retail building, tilt panel and metal roof.

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The buildings are all a similar type and are constructed of reinforced concrete and structural steel, with some timber framing. The standards I would expect an engineer to apply when designing this type of building include:

Standard Type	*Reference*	*Year*	*Application*	*Relevance*
Standards New Zealand	NZS 1170 Set Structural Design Actions	2002, 2011, 2003, 2016	Full Compliance	Used to access loads applied to the building, including gravity loads, earthquake loads and wind loads.
	NZS 3101 Concrete Structures Standard	2006	Full Compliance	Used to design concrete parts of the building.
	NZS 3404 Steel Structures Standards	2009	Full Compliance	Used to design steel parts of the building.
	NZS 3603 or NZS 3604	1993	Full Compliance	Used to design timber parts of the building..
Geotechnical Report	Project Specific		Full Compliance	Used to determine loads applied to soil
Fire Report	Project Specific		Full Compliance	Used to consider fire protection systems and post fire stability requirements.
MBIE Practice Advisory Notes	Bulletins	2005 - 2016	Be aware of and consider applicability	Industry Body information that highlights good practice.
SESOC, NZSEE, NZCS, HERA	Publications	Annual	Be aware of and consider applicability	Industry Body information that highlights good practice
Product Literature	Publications	Frequent	Be aware of and consider applicability.	Manufacturers information such as load span tables.

All the above reference documents are amended on an irregular basis because of building design problems or failures, new research, post-disaster observations or new knowledge in general. The CPD requirements of a chartered engineer require that he keeps up to date with such matters.

Current year of publication is shown in the table, however the documents have prior editions that would have been relevant at the time each building was designed. Where revisions to documents are relevant, this is discussed later in this report.

Tilt panels construction evolved as economical construction method over the last 15 years, and there have been several developments relating to tilt panel building construction. These developments stemmed from fire engineering issues where portal frames have dragged boundary panels into a building, or collapsed outwards, during a fire, presenting a danger to fire fighters or neighbouring properties. Other discussions stemmed from the discovery that NZ reinforcing bars can be brittle and intolerant to re-bending. Research has been carried out at Universities on panel base fixings. Some of the ensuing papers are listed below:

Body	*Author*	*Year*	*Title*	*Relevance*
NZSEE	B. J. Brown	1993	Some stability issues for tilt up precast panels under in plane seismic loading.	
NZCS	J. I. Restrepo and R. Park	1993	Review of Tilt-up Construction Details	.
SESOC	J. I. Restrepo, F. J. Crisafulli and R. Park	1996	Seismic Design Aspects for Tilt-up Buildings	
UOC?	Nigel Watts	1996	Tensile Capacity of Hooked Bar Anchorages with Short Embedment Lengths	
CCANZ		2004?	Tilt-up Technical Manual	A well circulated design reference.
NZSEE	B. J. Davidson	2004	Determination of seismic design forces for slender precast slab structures.	
Department of Building and Housing	R. A. Poole and M. S. Berkeley	2005	Report to DBH Review of Design and Construction of Slender Precast Concrete Walls	
BRANZ	G J Beattie	2007	Slender Precast Concrete Panels with Low Axial Load	
Reid	Reid Construction Systems	2007	Product Catalogue and Design Guides	
MBIE	John Gardiner	2013	Determination 2103/057	Are precast cladding panels parts.
	Joo H. Cho	2013?	Slender Precast Wall Panels	Industry Body information that highlights good practice.

SESOC	John Hare	2013	Considerations on Precast Panel Connections.	
NZSEE	P Armaos & D.M. Thomson	2016	Seismic Strengthening of a Commercial Warehouse with Slender Concrete panels	
MBIE	John Gardiner	2016	Determination 2016/003.	

As can be seen by comparing the dates of relevant papers and dates of the design of the Masterton buildings, there were significant developments in the design of such buildings over that period.

In completing this Initial Report, I have followed the following procedure:

1. Briefly reviewed each structural drawing for each project.
2. Tabulated initial comments.
3. Determined the likely load paths for each building.
4. Completed a rough assessment of seismic weight, seismic load and wind load.
5. Reviewed the primary members along the load paths to check strength.
6. Reviewed Connection Details.
7. Presented finding on a spreadsheet.

Tasks I have not completed at this stage due mostly to time limitations are:

1. Calculations submitted for building consent
2. Geotechnical reports
3. On site construction
4. Review of the peer review or consenting process
5. Discovery of Fire Reports

In making assessment I have based findings on simple hand calculations that I would expect an engineer to complete. I have not completed any computer modelling. Careful computer modelling may yield significantly better results than these calculations. I have assumed:

1. Soil Type C
2. No after fire load requirements.

Some of these buildings could conceivably flex and yield in an earthquake without sustaining catastrophic collapse. I refer to structures in Christchurch of this type which have been assessed as being <33%NBS which suffered only minor damage.

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Building Reference Name	*Roof Area*	*Mezzanine Area*	*Roof Height*	*Lateral Load System*	*Seismic Weight*	*Lateral Load*	*Comments*	*Conclusion*
Cnr Dixon and Church St	170	0	4.1	In plane PC walls, face loaded PC walls, timber walls, roof light bracing.	280	Vbx=78kN Vby=195kN	• Walls have significant out-of-plane and in-plane capacity • Walls are too flexible out-of-plane • Interaction with timber walls unclear • Connection at top of timber braced walls unclear • Lumberlock straps would be inadequate to act as bracing paths.	Lateral load systems look to be inadequate.
196-120 Queen St	290			X-Direction one legged portals fixing to PC panels Y-Direction roof bracing, wall bracing, in-plane PC panels	780		• One-legged portal frames supported by slender PC walls is an unusual lateral load resisting system • Slab thickening is unlikely to have sufficient weight to be effective	
57-65 (Lot 9) Dixon St	830			Connects to Existing Building X-Direction PC walls Grid A, bracing Grid B Y-Direction portals and existing wall grid 1	1300		• Critical bracing junction at Grids B4 is not detailed	Load paths in X-Direction are unclear. Footing detail (1/S21) to support a 7.4m 150 PC panel, that also serves as part of the portal is suspect.
57-65 Dixon St	450			X-Direction roof plane bracing and PC walls on grids 2 and 3 Y Direction PC walls on grids A and F, portals on grids C, D and E	1070		• Fixings to base of wall are fragile	This building appears to be better designed than others
[redacted]	[redacted]			[redacted].	[redacted]		• [redacted]	[redacted].
408 Queen St [redacted]	600			X-Direction in-plane roof bracing transfers load to grids A and B. Grid A is a wall, Grad B has a brace. Y-Direction has walls on Grids 1 and 4, portals on Grids 2 and 3. Walls on grid A appear to have been designed to cantilever.	660		• Roof bracing overstressed at mu=1.25, but OK if mu=3 • Wall brace grid B looks doubtful, 180 PFC is unlikely to restrain this brace, top connection to 180PFC inadequate • Possible ECC on SHS brace • Portals appear to lack compression flange braces • Wall on grid 4 prone to overturning • Precast panels vertical reinforcement on grid 1 inadequate • Panel over door on grid 1 dangerous fixings	Lateral load paths are clear for each direction. The panel over the door on Grid 1 is dangerous.

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LOAD PATHS

Load paths are evident on some buildings, but are unclear for 57-65 (Lot 9) Dixon St and [redacted] Lateral load resisting elements include roof bracing, in-plane PC walls and portal frames. The loads that occur along these load paths depend on assumed ductility and the building period. The assessments that I have made are generally based on mu=1.25 and T=0.4 which leads to the largest loads. If higher periods or ductilities can be justified, then ductile parts along the load paths could justifiably be designed for lower loads.

The rough calculation I have prepared, and review of drawings don't reveal the robust load paths that I would anticipate.

SLENDER PC PANELS

All projects utilise PC panels for walls and bracing elements.

Building Reference Name	*Year*	*Height / Thickness Slenderness Ratio*	*Comment*
Cnr Dixon and Church	2014	4.7 / 160 29	**Primary Face Loaded Lateral Load Resisting Element**
196-120 Queen St	2011	7.4 / 150 49	**Used as replacement portal leg – no roof plane bracing**
57-65 (Lot 9) Dixon St	2010	7.3 / 150 49	Restrained top and bottom
57-65 Dixon St	2006	7.3 / 150 49	Restrained top and bottom
[redacted]	[redacted]		[redacted]
408 Queen St [redacted]	2003	6.6 / 170 39	Restrained top and bottom. Lightly reinforced. **Unsafe panel over door.**

Panels are within commonly used slenderness ratios, however there are other concerning issues relating to these panels.

BASE FIXINGS TO PC PANELS

Building Reference Name	*Year*	*Fixing Description*	*Comment*

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Cnr Dixon and Church [redacted]	2014	Starter bars from large pads inserted into drossbachs.	Clearly used to provide cantilever capacity.
196-120 Queen St [redacted]	2011	1 row RB12@400 into 1500x200 slab thickening	Anchorage embedded beyond vertical reinforcement
57-65 (Lot 9) Dixon St [redacted]	2010	1 row RB16@400 into 2000x200 slab thickening	
57-65 Dixon St	2006	2 rows Y12@400 into 1500x300 slab thickening	Lower anchors near edge of panel.
[redacted]	[redacted]	[redacted]	
408 Queen St [redacted]	2003	1 row HD12@350 into 1750x250 slab thickening	

Anchors are required to provide base restraint, in-plane and gravity load transfer to the foundation slab thickening. The type of details that have been utilised on these building are similar to details shown in design guides. The anchorage of some of the small bars is questionable, however in many cases the loads on these anchors will also be low.

The attached appendices show and promote the types of footings adopted in these designs as being accepted practice.

TOP FIXINGS TO PANELS

Building Reference Name	*Year*	*Fixing Description*	*Comment*
Cnr Dixon and Church [redacted]	2014	90x8 EA, M12 C8 bolts at 600cs	No walls run parallel to purlins on this building.
196-120 Queen St [redacted]	2011	300 PFC eaves tie, nothing to low panels	
57-65 (Lot 9) Dixon St [redacted]	2010	250x90 PFC eaves tie, nothing to low panels	Bracing path to Grid B appears inadequate
57-65 Dixon St	2006	200 PFC eaves tie,	
[redacted]	[redacted]	[redacted]	
408 Queen St [redacted]	2003	No eaves tie grid A	The lack of eaves tie means little support is provided to the top of the wall.

Top fixings along precast panel lines are normally provided to support the top of panels, and transfer lateral loads to portal frames or end walls. They can also be important to connect to bracing or distribute loads back to the walls when they provide in-plane support.

The design of the transom requires judgement or determination of whether it is a building part, as defined by NZS1170, or is primary structure. Judgement is also required about the torsional stability of the transom is provided by the precast panel it supports. The load on the transom is related to the base fixity provided by the base fixings of the panel.

DID DESIGNS MEET ACCEPTABLE STANDARDS

Structural engineering designs vary significantly from one designer to the next. The variation is attributable to the background and experience of the engineer, as well as their emphasis, to either achieve economical designs or conservative designs. Some designers focus on safe designs regardless of the cost to the client, while others push the boundaries of low cost designs, while still using the codes as a minimum standard. There are also variations caused by the time available to complete a design and effort applied to the design.

The definition of meeting acceptable standards is difficult to define. My best analogy would be marking a project at Engineering School, it is inevitable that will be a wide variation in marks, and unlikely that anyone would score 100%. Even with the benefit of years of experience and an engineering registration process engineering knowledge and judgement varies.

I can only give a mark to the design of each building based on this review as follows:

Building Reference Name	*Year*	*Provisional Grade*	*Comment*
Cnr Dixon and Church [redacted]	2014	C	Cantilever panels would be too flexible to protect non-structural element
106-120 Queen St [redacted]	2011	C	Using slender PC panels as portal legs is not a robust load path.
57-65 (Lot 9) Dixon St [redacted]	2010	C+	Bracing load path at Grid B insufficiently detailed. Connection to existing building.
57-65 Dixon St [redacted]	2006	B	Appears to be a better design than others.
[redacted]	[redacted]	[redacted]	[redacted]
408 Queen St [redacted]	2003	D	Brace details, lack of eaves tie, lack of fly braces, light vertical wall reinforcement, and panel type C fixings all look deficient

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The designers of these building have clearly tried to meet acceptable standards while also developing low cost designs. Whether they have met acceptable standards is mostly a matter of judgement, but in some cases a specific element of the design can be highlighted and be proven to not comply with a specific clause of a standard.

Although these small structures appear relatively simple, to analyse them accurately would be a very complex task. This would involve computer modelling wall panels and their base fixity, stiffness of eaves channels and roof bracing elements. Such a complex analysis is usually avoided for practical reasons. Engineering judgement, and simplified calculations using tributary area and rational design processes are commonly used complete the design.

When carrying out this review I have worked in a reverse process. Starting with the drawings I have interpreted the load paths and likely loads. I have not carried the computer modelling that would be required to accurately assess the buildings.

Further detailed analysis may reveal the likely performance of the buildings in an earthquake, and validate my interpretation of the design, or possibly the original designers' assessment of loads.

Yours faithfully,

Stuart George
BGT Structures (Auckland) Limited

Appendices

Extract from BRANZ Design Guide
Extract from CCANZ Tilt-up Technical manual
Extract from Reid Design Guide

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For warehouse structures, the majority of designs will fall within either of the first two options (see earlier in this section for the exception). Of these, the first option is by far the most popular among designers, particularly because of the generally large available wall lengths.

Table 2. Ductility factor choice for design of buildings with slender precast wall panels

Structure type	Structural ductility factor, μ	Structural performance factor, S_p
Nominally ductile walls	1.25	0.9
Walls of limited ductility	3	0.7
Single-storey cantilevered face-loaded walls of limited ductility	2	0.8

6.1.1 Nominally ductile walls ($\mu = 1.25$, $S_p = 0.9$)

Walls that are designed as nominally ductile are expected to sustain only minor yielding damage in the design earthquake (one with an approximately 10% probability of exceedance in 50 years). As the earthquake strength increases beyond the design earthquake towards the maximum credible earthquake, nominally ductile designed structures will experience inelastic behaviour. For slender precast panels, this inelastic response will likely be yielding of the vertical flexural steel near the foundation interface. Experimental investigations [38][42][56] have shown that typical panels have been able to sustain actions well beyond the elastic range without collapse.

6.1.2 Limited ductile walls ($\mu = 3$, $S_p = 0.7$)

These walls will be designed for a force of approximately one-half of the force that would be used for an equivalent-sized wall designed to respond elastically. However, the actual potential earthquake will not be any less strong, and so it is expected that the limited ductile wall will sustain greater damage than the elastically responding wall when exposed to the same earthquake.

Within the wall, there is little scope to detail the reinforcing to cater for the greater damage. A single layer of steel cannot be confined with steel cages. Hence, it is expected that concrete spalling may occur at the ends of the wall base once the wall has passed its elastic limit and bar buckling may occur. For limited ductile walls, NZS 3101 requires (clause 11.4.4) that the sectional curvature ductility at the ultimate limit state be less than $15\alpha_{fy}$ times the nominal curvature at first yield, γ, where $\alpha_{fy} = 400/f_y$ (but not greater than 1.1), and

Figure 1 - Extract from BRANZ Design Guide (2007)

to the disparity of the moduli of elasticity and may corrode.

Spalling may occur if shims are located too close to the end of the panel, 300 mm is the recommended minimum distance. The size of the shims used should be determined by the load-carrying capacity of the shims and the bearing capacity of the concrete panel. They should not exceed 40 mm in height and should be not less than the width of the panel or 100 mm, whichever is less.

Other typical fixing and seating details are shown in **Figures 15 and 16.**

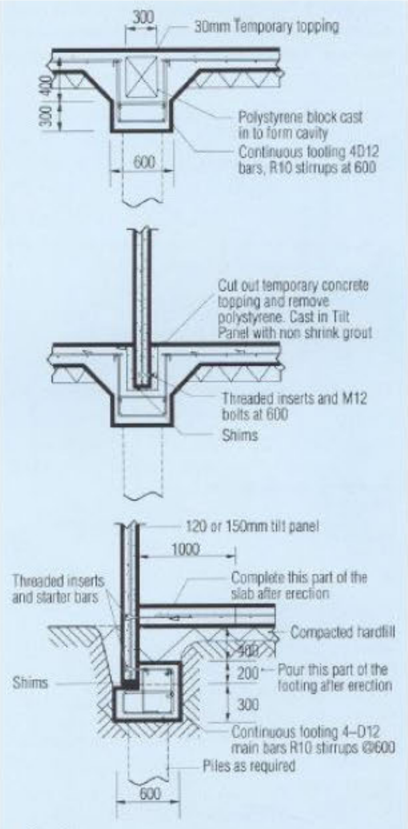

FIGURE 15
Cantilevered Party & Boundary Walls — Footing Detail

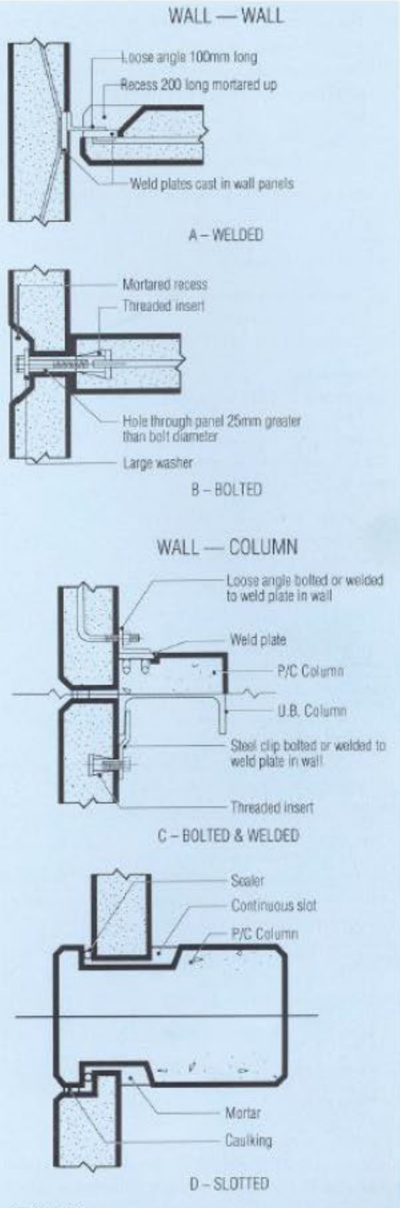

FIGURE 16
Some Typical Connections

Figure 2 - Extract from CCANZ Tilt-up Technical Manual

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Reid™ Design Concepts for Reinforcement Anchorage

reid™ CONSTRUCTION SYSTEMS

So how do I calculate concrete cone pullout ?

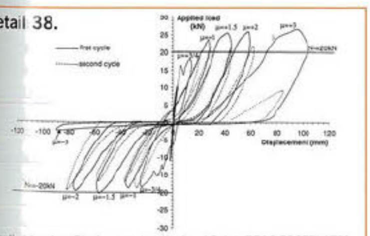
In 1993, NZ University of Canterbury research by *Restrepo-Posada and Park*[4] showed that the ψ-method can be used to predict the concrete cone capacity of hooked bar and headed stud type anchorages, provided that the correct embedment depth is defined. This design approach accounts for the influence of edge distance, bar spacing and micro cracking in tension zones, by applying reduction factors to the calculated concrete cone pull out capacity of the anchorage. To reduce the probability of premature brittle failures the approach also incorporates factors in the formula to account for likely variations in material strengths and construction tolerances.

The method and corresponding formula are set out in this manual in the form of a flow diagram on page 114 and is followed by a design example that compares the design of a "L" shaped hooked anchorage to that of a comparable sized Reid™ Threaded Insert anchor for a wall panel to foundation connection. (page 115.)

Importance of ductile failure

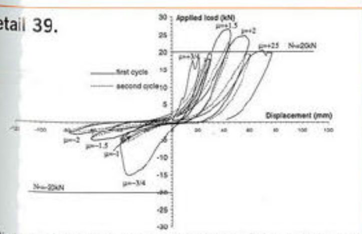
The importance of ductile failure should be appreciated, as it is essential to ensure that a brittle failure mechanism does not occur before a ductile failure, taking into account the possible material over strengths that can exist. In the design example it is shown that brittle failure of the anchorage will occur for both situations but in the case of the Threaded Insert it has enough capacity to ensure that the wall stem will have a ductile failure before cone pullout and thus provide a safe connection.

Detail 38.



Applied Load vs Displacement at the Load Point RB12 FOOTPLATES

Detail 39.

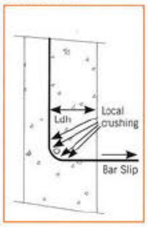


Applied Load vs Displacement at the Load Point RB12 BENT STARTERS

Anchorage slip

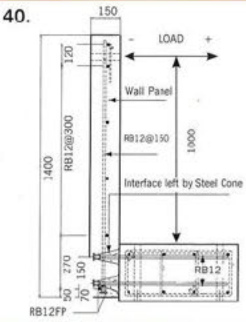
The ψ-method does not address slippage of the anchor. With hooked bars the inside of the hook causes local crushing of the concrete as the bar tries to straighten under load. Higher slippage of the reinforcing can occur compared to a headed anchor where the bearing stress under the head can be accommodated in the design of the product to minimize crushing.

Detail 37.

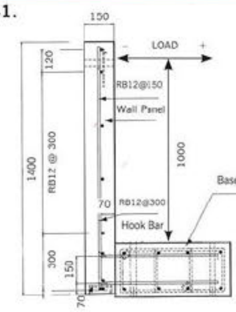


Research at NZ University of Auckland by *Maureen Ma in 1999 into Methods of Joining Precast Concrete components to form Structural Walls*[5] highlighted the performance of Reid™ Threaded Inserts compared to that of conventional hooked bar construction. The diagram below shows the test comparison between the two forms of anchorage in a wall panel to footing connection when subject to cyclic loading. It can be seen that the threaded inserts performed significantly better.

Detail 40.



Detail 41.



REIDBAR & FITTINGS

Figure 3 - Extract from Reid Design Guide

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12362

19 April 2018

IPENZ
Level 3
50 Customhouse Quay
Wellington,
By email

Attention: Investigator

Dear Sir,

Re: Masterton Inquiry

In response to matters raised in your letter of 11 April 2018:

1. Quality Assurance (QA) processes are internal processes that are not generally mandated but are good professional practice.
2. QA processes have evolved in a formal way from about 1990 when ISO 9000 procedures were being promoted to the building industry. Some engineering practices around that time obtained ISO 9000 accreditation.
3. Consulting engineers are usually legally contracted to exercise reasonable skill, care and diligence in the performance of their duties. They also have an ethical obligation to act in a careful and competent manner.
4. IPENZ/ACENZ introduced a Practice Note on Structural Engineering Design Office Practice circa 2009 (earlier versions exist in ACENZ Practice Notes) stating some form of quality assurance or internal review process is essential to ensure consistent defect-minimised design output.
5. Old school "Ministry of Works" practices required all drawings to be signed by the Designer and by the Checker. Designer and Checker would be an Engineer and a Senior Engineer. This was an early form of QA.
6. Good office practice would require each project to be assessed based on size, complexity, designer experience, and difficulty. From that assessment an appropriate level of design verification would be introduced. (refer Appendix for BGT Structures system). Record keeping of QA systems is a key part of the process.

Concept design could be carried out by anyone who is competent. Having a CPEng would not necessarily ensure an engineer is competent although it is a good indicator that he may have the necessary

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knowledge. Hence, I do not see any issue with a concept being carried out by a senior engineer. However, I would expect the Concept Design prepared by the senior non-CPEng engineer to be reviewed by the engineer who was intending to sign a PS1. The procedure KOA have described seems to omit who actually does the developed design and detailed design steps, which occur between concept design and the senior engineer checking calculations.

It is normal for a technician to do all drawing development work.

The Masterton projects under investigation would require a Level 1 verification using our office procedures. Having a senior non-CPEng engineer check the calculations and drawings, followed by a CPEng engineer carrying out a high-level review of the design and calculations would be a suitable Level 1 QA procedure.

Over the period that the Masterton projects were designed we would expect that QA Practice would have evolved and improved in most engineering offices. We would also expect some variance between large and small engineering practices. From my experience, the table below could be used as a guide to the evolution of QA procedures. It is likely that there is considerable variation in QA procedures between different engineering offices in New Zealand. It is not possible to accurately define reasonable practice, as it evolved on a yearly basis as requested.

Period	QA System	Peer Review System (not part of Design Office QA System)
2010-present *(following Open Letter, Canterbury Earthquakes and Royal Commission)*	Larger practices have formally documented QA processes and records are kept. Project QA meetings are used to gain input from a broader knowledge base. Smaller practices may still rely on self-checking.	Some TA's requiring evidence of office QA procedures. All large projects subject to independent peer review. Peer review scope well described. Peer review logs being kept.
1980-2010 *(codes became more complex, buildings became more complex, new design procedures were developing)*	ISO 9000 procedures adopted and encouraged. Little record keeping of QA procedures. Senior engineers and Directors checking designs. Producer Statements introduced requiring formal sign-off. Practice notes setting out formal office procedures and check lists.	Many TA's required designs to be submitted to an independent Engineer for checking – often appointed by TA. Variable procedures amongst different TA's.
Pre 1980's *(buildings were generally simpler)*	Designer and Checker to sign all drawings. Time spent at end of project design on checking.	Many TA's had own Engineers who checked designs

The table sets out comments on the described QA procedure for each of the Masterton buildings. We have split the design and QA procedures into the following steps:

a) Concept Design
b) Developed Design and Detailed Design (calculations and drawings)
c) Drawings and Calculations checking by a senior engineer.
d) High-level review and issue of a PS1 by CPEng engineer.

Building	Structural issue	KOA Peer Review Process Responsibility Step
[redacted] *(2014)* Cnr Dixon and Church Sts	• Walls have significant out-of-plane and in-plane capacity • Walls are too flexible out-of-plane • Interaction with timber walls unclear • Connection at top of timber braced walls unclear • Lumberlock straps would be inadequate to act as bracing paths.	a) b), d) c), d) b), d)
[redacted] *(2011)* 196-120 Queen St	• One-legged portal frames supported by slender PC walls is an unusual lateral load resisting system • Slab thickening is unlikely to have sufficient weight to be effective	a), d) b), d)
[redacted] *(2010)* 57-65 (Lot 9) Dixon St	• Critical bracing junction at Grids B4 is not detailed	b), c)
[redacted] *(2006)* 57-65 Dixon St	• Fixings to base of wall are fragile	c)
[redacted]	• [redacted]	[redacted]
[redacted] *(2003)* 408 Queen St	• Roof bracing overstressed at mu=1.25, but OK if mu=3 • Wall brace grid B looks doubtful, 180 PFC is unlikely to restrain this brace, top connection to 180PFC inadequate • Possible ECC on SHS brace • Portals appear to lack compression flange braces • Wall on grid 4 prone to overturning • Precast panels vertical reinforcement on grid 1 inadequate • Panel over door on grid 1 dangerous fixings	b), c) b), c), d) d) c), d) c) c) b), c), d) PS1 should not have been signed

Although we have noted the PS1 should not have been signed on two projects, we don't want to imply the PS1 should have been signed on the remaining four projects. Please refer to our Initial Report of 11 August 2017 which sets out a grade for each building.

[redacted]

[redacted]

We trust this letter adequately responds to your request.

Yours faithfully,

Stuart George
BGT Structures (Auckland) Limited

22. Appendix 1 - Pre-Engagement Capability and Risk Assessment Pro-Forma

Assessment by – *Stuart George* Date – *1 July 2016*

Criteria	Assessment				Score
Project Fee	Small <\$30k 1	Medium \$30-100k 3	Large \$100-500k 6	Major >\$500k 10	*10*
Project Difficulty	Routine 1	Difficult 2	Complex 4	Ground-Breaking 6	*4*
Designer Experience	Low[1] 8		Medium 3	High[2] 1	*1*
Project Programme	Relaxed 1	Average 2		Compressed 4	*2*
Liability Accepted	Low 0	Standard 1		Severe[4] 4	*4*
Job Profile	Low 1	Medium 2		High[5] 4	*4*
				Total Score	***25***

1 Less than 18 months experience
2 CPEng or equivalent
3 Risk transferred
4 Unfavourable job-specific conditions
5 Of national importance/renowned

Level 1 Design Verifications	Monitor output occasionally. Project Director check of drawings at release	Score 5 - 10
Level 2 Design Verifications	Formal design team review plus Project Director interim check. DFR required.	Score 11 - 15
Level 3 Design Verifications	Formal internal peer review with check on design concept and philosophy, detail check. DFR Required.	Score 16 - 20
Level 4 Design Verifications	*As per Level 3 Design Verification plus a check of representative calculations and details. DFR Required*	*Score 21 - 25*
Level 5 Design Verifications	External peer review. (Include at least Level 3 Design Verification prior to handing over for external review. DFR Required	Score over 25

****Highlight design verification level.*