UNDER	THE COMMISSIONS OF INQUIRY ACT 1908
IN THE MATTER OF	ROYAL COMMISSION OF INQUIRY INTO BUILDING FAILURE CAUSED BY CANTERBURY EARTHQUAKES KOMIHANA A TE KARAUNA HEI TIROTIRO I NGA WHARE I HORO I NGA RUWHENUA O WAITAHA
AND IN THE MATTER OF	THE CTV BUILDING COLLAPSE

THIRD STATEMENT OF EVIDENCE OF ASHLEY HENRY SMITH IN RELATION TO INTERPRETATION OF DESIGN CODES FOR COLUMNS AND BEAM-COLUMN JOINTS INTHE CTV BUILDING

DATE OF HEARING: COMMENCING 25 JUNE 2012

THIRD STATEMENT OF EVIDENCE OF ASHLEY HENRY SMITH IN RELATION TO INTERPRETATION OF DESIGN CODES FOR COLUMNS AND BEAM-COLUMN JOINTS INTHE CTV BUILDING

INTRODUCTION

1. My name is Ashley Henry Smith. I live in Auckland. I am the director of StructureSmith Ltd, a consulting engineering company specialising in structural engineering.

QUALIFICATIONS AND EXPERIENCE

2. My qualifications and experience are outlined in my first statement of evidence dated 27 April 2012.

EVIDENCE

3. I have been asked to provide evidence to the Canterbury Earthquake Royal Commission relating to my interpretation of the structural design codes of the day i.e.:
   - NZS4203:1984 The Code of Practice for General Structural Design and Design Loadings for Buildings; and
   - NZS3101:1982 The Code of Practice for Design of Concrete Structures

   as they applied to the design of columns and beam-column joints in the CTV Building, in particular:

   3.1. The requirement to design the columns to possess ductility; and

   3.2. The requirement to design the columns to withstand deformations due to earthquake loads.

4. I have read and agree to comply with the Code of Conduct for Expert Witnesses, a copy of which is attached and marked **"A"**.

5. I confirm that the matters I am giving evidence about are within my areas of expertise.

CODE INTERPRETATIONS VARIED

6. Interpretations of these code requirements varied between myself and Dr Clark Hyland, the co-authors of the CTV Building Collapse Investigation report dated 27 January 2012 (the Hyland/Smith report) as stated on page 12 of that report. The interpretation described on page 20 and page 109 of the Hyland/Smith report, and in the Appendix F titled 'Displacement Compatibility Analysis to Standards' was Dr Hyland's interpretation.

7. It is worth noting that other members of the Department of Building and Housing Expert Panel also had different interpretations of the codes in relation to column design. This is an indication to me that the design codes of the day were not entirely clear.

8. Although there were variations of interpretation, neither myself nor Dr Hyland and also none of the DBH Expert Panel members thought that the design of the columns would have complied. The variations of interpretation were about the extent of non-compliance only and which of the criteria in the codes was most critical. My interpretation of the design codes follows.

THE REQUIREMENT TO DESIGN THE COLUMNS TO POSSESS DUCTILITY

9. The CTV Building was designed with the reinforced concrete North Core walls and the South Wall as the primary bracing system to resist lateral loads from earthquake. These walls were designed to be ductile and were therefore required to be designed to be capable of dissipating seismic energy by flexural yielding.

10. Further, the North Core walls and the South Wall were required to be subject to capacity design, which is defined in NZS3101:1982 as follows:
"CAPACITY DESIGN (definition). In the capacity design of earthquake resistant structures, elements of the primary lateral load resisting system are chosen and suitably designed and detailed for energy dissipation under severe deformations. All other structural elements are then provided with sufficient strength so that the chosen means of energy dissipation can be maintained."
In the case of the CTV Building "all other structural elements" would have included the columns.

11. There was an overriding requirement in clause 3.2.1 of NZS4203:1984, that was applicable for all buildings, i.e. not only those constructed of concrete that stated:

    "The building as a whole, and all of its elements that resist seismic forces or movements, or that in case of failure are a risk to life, shall be designed to possess ductility:….."

    The CTV Building columns were elements that had to resist seismic movements and they were also a risk to life if they failed. Therefore, under this clause 3.2.1 they were required to be designed to possess ductility.

12. As defined in NZS4203:1984:

    "DUCTILITY means the ability of the building or member to undergo repeated and reversing inelastic deflections beyond the point of first yield while maintaining a substantial proportion of its initial maximum load carrying capacity."

13. Applying this definition to the CTV Building columns, I would interpret it to mean there was a requirement for the columns to be able to undergo repeated and reversing inelastic horizontal deflections (or inter-storey drifts) beyond the point of first yield while maintaining a substantial proportion of their initial maximum vertical load carrying capacity.

14. Circular concrete columns are generally designed to possess ductility by providing a number of vertical steel bars around the perimeter of the section, and wrapping around those vertical bars a sufficient quantity of spiral reinforcing, spaced at sufficiently close centres so as to:

    14.1. prevent 'shear' failure and

    14.2. provide 'confinement' to the concrete core of the column to ensure that it can maintain vertical load-carrying capacity.

15. There are separate sections in the Code of Practice for the Design of Concrete Structures NZS3101:1982 for design for 'shear' (section 7), and for design for flexure and axial load including 'confinement' (section 6). Shear is generally considered separately from confinement in the design of reinforced concrete columns. Spiral reinforcement is required to resist shear and to provide confinement, but according to different rules for different regions of each column. The final design for each column needs to have at least the minimum quantity of spiral reinforcement required for shear or for confinement at each section, whichever quantity is greatest.

THE REQUIREMENT TO PREVENT SHEAR FAILURE

16. 'Shear' force in a column refers to the horizontal force, which is generally constant over the height of each column at each storey in the building. If shear forces become large they can result in the formation of inclined cracks, followed by shear failure if those inclined cracks become too large or separate altogether. This is a non-ductile or brittle type of failure which is to be avoided. Therefore an essential first step in the design of a column is to ensure sufficient shear strength by including an appropriate minimum quantity of spiral reinforcement to provide tension across any potential inclined shear cracks, thereby preventing such cracks from opening.

17. The minimum requirements to prevent shear failure of columns were clear in the standards of the day for the CTV Building and there was no variation of interpretation on this aspect by myself, Hyland or any others in the DBH Expert Panel.

18. NZS3101:1982 clause 7.3.4 states that "A *minimum area of shear reinforcement shall be provided in all reinforced … concrete where shear stress vi required to resist Vu exceeds half the shear strength provided by concrete …* ", where

    18.1. vi is the total; shear stress and

    18.2. Vu is the factored shear force at the section

19. NZS3101:1982 commentary clause C7.3.4 provides further guidance as follows: *"When repetitive loading might occur on flexural members the possibility of inclined diagonal tension cracks forming at appreciably smaller stresses than under static loading should be taken into account in the design. In these instances, it would be prudent to use at least the minimum shear reinforcement … even though tests and calculations based on static loads show that shear reinforcement is not required."*

20. My interpretation of this NZS3101:1982 clause 7.3.4, taking into account the point in items 11 and 12 above that the there was a requirement for the columns to be able to undergo repeated and reversing inelastic horizontal deflections, is that at least the minimum shear reinforcement was required over the full height of all columns. I calculated the minimum shear reinforcement as R6 spiral @ 90 mm centres or R10 spiral @ 150 mm centres, as explained on page 110 of the Hyland/Smith report.

THE REQUIREMENT TO PROVIDE CONFINEMENT

21. There were variations of interpretation by the authors and by others on the DBH Expert Panel over the requirements for confinement of columns in NZS3101:1982. The reason why different interpretations existed can be explained partly by the structure of the standard, as explained in the Foreword to the standard which is attached and marked **"B"**.

22. Paragraphs 3 and 4 of the Foreword state:

"The arrangement of clauses represents a significant change in format from the previous code with the aim of producing a more workable document.
The intended order of usage is that after proceeding through Notation, Scope and General principles and requirements which apply to all structures, the designer then goes either to: Principles and requirements additional to Clause 3 for members <u>not</u> designed for seismic loading, or to: Principles and requirements additional to Clause 3 for members designed for seismic loading, that is, only one of the last two clauses is used, not both…" (see diagram attached).

23. If we consider the above arrangement of clauses in relation to section 6 of the standard, where the provisions for confinement for columns are contained, we see that there are no general requirements for confinement under section 6.3, and there are different requirements for spiral confinement in clause 6.4.7.1 the additional requirements for members *not* designed for seismic loading, and under clause 6.5.4.3 the additional requirements for members designed for seismic loading. Also, within clause 6.4.7.1 there is the option for using either a strength reduction factor of 0.9 (clause 6.4.7.1 (a) or a strength reduction factor between 0.7 and 0.9 (clause 6.4.7.1.(b) each with different requirements for minimum confinement.

24. Further, under certain conditions structures and members, including columns, could be designed for 'limited ductility', in which case the additional requirements of section 14 of the standard would apply. The provisions for minimum confinement for columns in section 14 are contained in clauses 14.6.2 and 14.6.3.

25. The rationale for the limited ductility section 14 in the standard is explained at the bottom of page 12 of the Foreword attached as follows:

"Section 14 gives the design and detailing provisions for members in structures of limited ductility subjected to earthquake induced loading. This section recognises

that less stringent ductility requirements are appropriate because of the larger lateral design loads applicable to such structures."

26. Thus, if one is designing confinement for a column in accordance with NZS3101:1982, the requirements for the minimum amount of spiral reinforcement are contained in four separate clauses in the standard, as follows

    26.1. 6.4.7.1 (a) under the additional requirements for members *not* designed for seismic loading, when a strength reduction factor of 0.9 is used

    26.2. 6.4.7.1 (b) under the additional requirements for members *not* designed for seismic loading, when a strength reduction factor between 0.7 and 0.9 is used

    26.3. 6.5.4.3 under the additional requirements for members designed for seismic loading, with full ductility, and

    26.4. 14.6.2 and 14.6.3 under the additional requirements for members designed for seismic loading, with limited ductility.

27. It is evident from a review of the structural drawing S14 titled 'Columns' for the CTV Building that confinement for those columns was designed on the basis if the minimum requirements to NZS3101:1982 clause 6.4.7.1 (b) only, because it would not satisfy any of the other three clauses referred to in item 26 above.

28. Considering the clauses listed in item 26 above, it would have been sufficient to detail the columns for the CTV Building with full ductility in accordance with clause 6.5.4.3 without any further checks. However, for any of the other three options for design of confinement, further checks were required to ascertain which clauses were appropriate and which were not appropriate.

29. If we start by considering the choice between clauses 6.4.7.1 (a) and 6.4.7.1 (b) it is helpful to read the commentary NZS3101:Part2:1982 clause C6.4.7. to understand the rationale behind these code provisions. The commentary is relevant because, as explained in the Foreword to the standard attached:

    "A comprehensive commentary is published with the code and it is strongly recommended that the two documents should be read together."

30. Commentary clause C6.4.7 states:

"Columns may be designed using a strength reduction factor ø of 0.9 … if the quantity of and arrangement of transverse reinforcement is adequate to ensure ductile behaviour. Clause 6.4.7.1 (a) specifies the required spiral or circular hoop reinforcement .. considered necessary for using ø = 0.9. The amount of spiral reinforcement required by eq. 6-3 (i.e. clause 6.4.7.1 (a)) is intended to provide additional load-carrying strength for concentrically loaded columns equal to or slightly greater than the strength lost when the shell spalls off."

31. The rationale behind clause 6.4.7.1 (a) is therefore consistent with the requirement of NZS4203:1984 clause 3.2.1 for the columns to possess ductility, and the definition of ductility in that standard, as outlined in items 11 and 12 above.

32. In item 30, the 'shell' refers to the cover concrete around the perimeter of the column section outside the line of the reinforcement. This is particularly important for the CTV Building columns because they are relatively small 400 mm diameter sections with 50 mm concrete cover to the inside face of the spiral, and so the concrete core contained by the spiral is only around 56% of the total section area. Consequently the strength lost when the concrete shell spalls off is a significant proportion of the total strength.

33. The final paragraph of commentary clause C6.4.7 is also relevant and that states:

"Note that when the axial load on the column is low (that is, P_u/f'_cA_g is relatively small), the dependable strength of the column with transverse steel for low ductility (ø < 0.9) may be adequate. However the ductile design case (ø = 0.9) is a useful means of increasing the dependable strength of the column when the axial load on the column is relatively heavy."

34. This is important in relation to the CTV columns because the axial loads in the columns are relatively heavy (the ratio P_u/f'_cA_g varies up to 0.58). In relation to the CTV Building my interpretation of the last paragraph of clause C6.4.7 is that the axial loads on the columns were high and so the dependable strength of a column with transverse steel for low ductility (ø < 0.9) may *not* be adequate. For this reason I consider that design of confinement based on the lower limit of NZS3101:1982 clause 6.4.7.1 (b) for members not designed for seismic loading, and with a strength reduction factor between 0.7 and 0.9 would not be appropriate for the CTV Building.

35. NZS3101:1982 clause 3.5.14.3 lists requirements for Group 2 secondary structural elements, which include the columns of the CTV Building. The first paragraph in this clause states that

"Group 2 elements shall be detailed to allow ductile behaviour and … "

This statement is consistent with NZS4203:1984 clause 3.2.1 which required the columns to be designed to possess ductility, as outlined in item 11 above. We also need to keep in mind the definition of 'ductility' outlined in item 12 above, in particular the need for the columns to maintain a substantial proportion of their initial maximum load carrying capacity.

36. NZS3101:1982 clause 3.5.14.3 then goes on to outline the various conditions where the additional seismic provisions of the code need or need not be satisfied, including (f) when the requirements of section 14 for limited ductility may be applied. However, in my view, for the reasons explained in item 34 above, design of confinement for columns based on the lower limit of NZS3101:1982 clause 6.4.7.1 (b) for members not designed for seismic loading, and with a strength reduction factor between 0.7 and 0.9 would not be appropriate in any case under clause 3.5.14.3.

37. Overall, my interpretation of the minimum requirements for spiral reinforcement in the CTV Building columns according to the standards of the day would be:

37.1. R10 @ 150 mm spiral in the mid-height regions of columns, between the potential plastic hinge regions (governed by shear), and

37.2. R10 @ 75 mm *or closer* spacing spiral in the potential plastic hinge regions at the top and bottom of each column at each storey (governed by confinement). The 75 mm spacing would be appropriate only if design in accordance with section 14 for limited ductility was applicable. That would be debatable in my view, but may be relevant for the lighter loaded columns in the upper levels.

38. I calculated the confinement that would have been required in the potential plastic hinge regions of the 'indicator columns' in the Hyland/Smith report according to

NZS3101:1982 clause 6.5.4.3 (the additional requirements for members designed for seismic loading, with full ductility) as follows:

38.1. For the indicator column at grid F2 level 3, R10 spiral at 50 mm spacing, and

38.2. For the indicator column at grid D2 level 3, R10 spiral at 40mm spacing

This is recorded in my email dated 3 February 2012 attached and marked **"C"**.

39. I also calculated the confinement that would be required in the potential plastic hinge regions of the 'indicator columns' in the Hyland/Smith report according to NZS3101:1982 clause 6.4.7.1 (a) (the additional requirements for members *not* designed for seismic loading, when a strength reduction factor of 0.9 is used) and found that to be more onerous than the confinement calculated in item 38 above.

40. It is informative to read the paper titled "Evaluation of a 10-Storey Building using Alternative Structural Systems" written by D K Bull, who at that time was Structural Engineer for the Cement & Concrete Association of New Zealand in October 1991. This paper was written after the CTV Building was designed and so would not have been available to take account of in the design. It is referred to here only to demonstrate the debate that had occurred about some of the NZS3101:1982 code provisions. The paper by Bull is attached and marked **"D"**.

41. Section 4.2.4 on page 11 in the paper by Bull is titled Design Philosophies: Gravity Frames and in this section Bull discusses methods of analysis and interpretations of NZS3101:1982 clause 3.5.14 for secondary structural elements. The first paragraph in 4.2.4 states:

At the inception of the study there was much debate as to whether "gravity" frames should or should not be carrying lateral load? And were the designs to be "limited ductility" approaches or "full ductility" approaches.

42. Also in the paper by Bull, under section 4.2.4.2 titled Gravity Frames: Full Ductility of Limited Ductility?:

"The first reaction after deciding that the gravity structures were "secondary" was to start designing on the basis of "members not designed for seismic loading … and go to a limited ductility approach. This proved to be not completely appropriate."

43. My interpretation of these statements in the paper by Bull is that there would have been debate about the various options for detailing confinement to gravity frames, including columns, under the rules of NZS3101:1982 as outlined in item 26 above.

44. In relation to NZS3101:1982 clause 6.4.7.1 (b), my interpretation of the 1991 paper by Bull is that particular clause in the standard was intended only for 'gravity frames with negligible lateral load capacity' as per section 4.2.4.3 by Bull, or for the mid-height regions of columns between potential plastic hinge regions as per section 4.2.4.4 by Bull.

45. In the case of the CTV Building this would be consistent with my interpretation, as outlined in items 34 and 36 above. However, in my opinion NZS3101 could have provided further explanation to simplify and clarify the intended uses and limitations for clause 6.4.7.1 (b), if that was the intention.

46. It is significant that the structure of the standard NZS3101 was changed from 1995, as explained in the Foreword to NZS3101:1995 attached and marked **"E"** as follows: *This standard features an organisational structure which is essentially the same for NZS3101:1982. However, for the majority of sections which contain seismic provisions, there is no longer a separate clause covering the requirements for member/structures not designed for seismic forces. Such requirements are now included in clause X.3, General principle and requirements for design, with seismic provisions being addressed in clause X.4, Additional design requirements for earthquake effects.*

47. Accordingly, one of the key changes in 1995 was that minimum requirements for confinement for columns were stated in the General section, thereby removing the options outlined in item 26 above in relation to minimum requirement for confinement. The other key change was that the option to use a strength reduction factor between 0.7 and 0.9 and the associated reduced quantity of confinement was removed.

BEAM-COLUMN JOINTS

48. According to NZS3101:1982 clause 9.4.8

"The horizontal transverse confinement reinforcement in beam-column joints shall not be less than that required by 6.4.7, with the exception of joints connecting beams

at all four column faces in which case the transverse joint reinforcement bay be reduced to one half that required in 6.4.7, but in no case shall the stirrup tie (or spiral) spacing in the joint core exceed ten times the diameter of the column bar or 200mm, whichever is less."

49. None of the CTV Building columns have beams connecting at all four faces and so the confinement had to be not less than that required by 6.4.7. Because the beam-column joints are an integral part of the columns my interpretation is that it would have been necessary to provide confinement in accordance with 6.4.7.1 (a) and not 6.4.7.1 (b), for the same reasons explained in item 34 above.

Signed: ....

ASHLEY HENRY SMITH

Date: 21 May 2012

Statutes of New Zealand

High Court Rules

Schedule 4

Code of conduct for expert witnesses

r 9.43

Duty to the court

1 An expert witness has an overriding duty to assist the court impartially on relevant matters within the expert's area of expertise.

2 An expert witness is not an advocate for the party who engages the witness.

Evidence of expert witness

3 In any evidence given by an expert witness, the expert witness must—

(a) acknowledge that the expert witness has read this code of conduct and agrees to comply with it:

(b) state the expert witness' qualifications as an expert:

(c) state the issues the evidence of the expert witness addresses and that the evidence is within the expert's area of expertise:

(d) state the facts and assumptions on which the opinions of the expert witness are based:

(e) state the reasons for the opinions given by the expert witness:

(f) specify any literature or other material used or relied on in support of the opinions expressed by the expert witness:

(g) describe any examinations, tests, or other investigations on which the expert witness has relied and identify, and give details of the qualifications of, any person who carried them out.

4 If an expert witness believes that his or her evidence or any part of it may be incomplete or inaccurate without some qualification, that qualification must be stated in his or her evidence.

5 If an expert witness believes that his or her opinion is not a concluded opinion because of insufficient research or data or for any other reason, this must be stated in his or her evidence.

Duty to confer

6 An expert witness must comply with any direction of the court to—

(a) confer with another expert witness:

(b) try to reach agreement with the other expert witness on matters within the field of expertise of the expert witnesses:

(c) prepare and sign a joint witness statement stating the matters on which the expert witnesses agree and the matters on which they do not agree, including the reasons for their disagreement.

[7 In conferring with another expert witness, the expert witness must exercise independent and professional judgment, and must not act on the instructions or directions of any person to withhold or avoid agreement.**]**

History Note - Statutes of New Zealand

Clause 7 was substituted, as from 1 December 2009, by r 10 High Court Amendment Rules (No 2) 2009 (SR 2009/334).

History Note - Statutes of New Zealand

The High Court Rules were substituted, as from 1 February 2009, by s 8(1) Judicature (High Court Rules) Amendment Act 2008 (2008 No 90). See s 9 of that Act for the transitional provisions.

FOREWORD

The objectives in drafting this Code NZS 3101 Part 1 and its commentary NZS 3101 Part 2 have been to provide an up-to-date design code which covers the design of buildings, bridges and other civil engineering structures. In writing all the sections, particular attention has been given to producing provisions which would be appropriate for use with the modern New Zealand design loading codes – particularly with NZS 4203:1976, *Code of Practice for general structural design and design loadings for buildings.* The Code is a revision of NZS 3101P:1970 and it has been extended to cover the design requirements for prestressed concrete. Concurrently with the publication of this document, NZS 3101P:1970 and NZSR 32:1968 *Prestressed concrete,* are revoked.

Generally the design requirements of each section of the Code are presented under five clauses in the following order:

Clause 1	Notation
Clause 2	Scope
Clause 3	General principles and requirements for design
Clause 4	Principles and requirements additional to Clause 3 for members *not* designed for seismic loading
Clause 5	Principles and requirements additional to clause 3 for members designed for seismic loading.

This arrangement of clauses represents a significant change in format from the previous code with the aim of producing a more workable document.

The intended order of usage is that after proceeding through Notation, Scope and General principles and requirements which apply to all structures, the designer then goes either to: Principles and requirements additional to Clause 3 for members *not* designed for seismic loading, or to: Principles and requirements additional to Clause 3 for members designed for seismic loading, that is, only *one* of the last two clauses is used, not both. (See diagram below.)

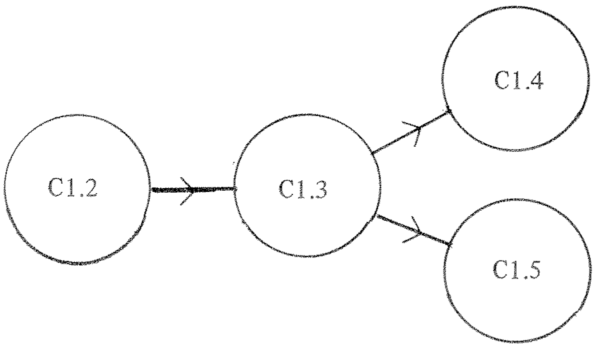

DIAGRAM INDICATING ORDER OF USAGE OF CLAUSES

Section 3, General design requirements, has a particular importance in the Code for two reasons:

(a) It covers the use of all other sections which should not be used in isolation, but should be read together with Section 3

(b) It establishes the relationship of this Code to the Loadings Code NZS 4203 and to the Ministry of Works and Development Highway Bridge Design Brief.

It should be noted that some provisions in this Code are based on proposed amendments to NZS 4203 which at the time of publication are being finalized.

Section 14 gives the design and detailing provisions for members in structures of limited ductility subjected to earthquake induced loading. This Section recognizes that less stringent ductility requirements are appropriate because of the larger lateral design loads applicable to such structures.

The Code permits considerable simplification in design procedures to be achieved if a structure is treated as responding elastically to earthquakes, under the provisions of 3.5.1.1 (c). This exempts the structure from the additional seismic requirements of all relevant sections of the Code. There will be many small structures, and some structural forms having substantial total lengths of wall in each direction, where the larger design seismic loads required for elastically responding structures will not result in significant cost increase. Alternatively, significant simplification can be obtained by the use of the procedures for design of structures of limited ductility set out in Section 14.

With the exception of the provisions for seismic loading, ACI 318-77 *Building code requirements for reinforced concrete,* has been used with minor modification. Following the practice of ACI 318-77 all sections commence with a list of notation used in that section. In addition, a list of the entire set of symbols used in the Code is presented in Appendix A. It should be noted that some symbols can have different meanings in different sections.

Appendix B presents an alternative design method which is based on working stress design whereas the main body of the code is based on the strength method of design with serviceability checks. In particular the strength method of design is mandatory for seismic design.

A comprehensive commentary is published with the code and it is strongly recommended that the two documents should be read together. This commentary is presented in some length with the aim of providing guidelines without unnecessary restriction. The appendix to Commentary Section 3 (C3.A) "A method for the evaluation of column action in multistorey frames" is a special example of this intention. This appendix is included to give designers guidance in the assessment of the maximum actions on columns resulting from capacity design considerations. Because of its developmental stage and as it is possible to use other methods, it is not a mandatory provision. At the end of several commentary sections a list of references is provided to assist designers in areas where standard design procedures have not yet been formulated.

"C"

From: Ashley Smith (StructureSmith)
To: "David Hopkins"; "Clark Hyland"; "Sherwyn Williams"; "Nigel Priestley"; "Rob Jury"; "Adam Thornton"; "Vicky Newton"; "Dr Helen Anderson"; "Stefano Pampanin"; "Peter Fehl"; "Peter Millar"; "George Skimming"; "Marshall Cook"
Cc: "Avon Adams"; "Ann Clark"; "Kate Ryder"; "Sarah Morton"; "Raphael Hilbron"; "Vicky Newton"; "Neil Green"; "Nerys Parry"; "Mike Stannard"; "David Kelly"
Subject: RE: CTV Presentation Updated to 2 Feb for your information - confinement steel in accordance with NZS 3101:1982 section 6.5
Date: Saturday, 1 January 4501 12:00:00 a.m.
Attachments: column confinement to NZS3101-1982 section 6-5.pdf

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David, and others

Please find attached details of the spiral confinement that would have been required in accordance with NZS 3101:1982 section 6.5 (ductile members designed for seismic loading i.e "the additional seismic requirements of the code") for the two indicator columns described in the CTV and Panel reports. These are the column at grid F2 level 3 and the column at grid D2 level 3. The required spiral has been marked in red on an extract from the original structural drawing, alongside the R6@250mm spiral that was detailed. Points to note as follows:

1. R6 spiral would not comply, because it would need to be placed at approximately 15mm centres and therefore would not comply with the required minimum clear spacing of 25mm.
2. I have shown R10 spiral, which has almost 3 times (actually 2.77 x) the steel cross section area for each bar and can therefore be placed at the wider (complying) spacings shown on the attached mark-up.
3. The required confinement varies slightly depending on the axial load in the column. Column D2 caries more axial load than column F2 and therefore required more spiral, or in this case the same spiral at closer spacing, as indicated on the attached mark-up.
4. The length of the potential plastic hinge zones also varies depending on the level of axial load. For column F2 the potential plastic hinge length is D = 400mm, and for column D2 the length is 1.5D = 600mm. These are the zones where the most tightly spaced spiral would have been required, as indicated on the attached mark-up.
5. The tightest spiral spacing would also apply through the beam-column joint zones, as indicated on the attached mark-up, although a similar quantity of separate hoops would normally be used in the joint zones because that would be more practical given the nature of the construction with insitu columns and precast beams.
6. The code required that "over the length of the column adjacent to the potential plastic hinge region the quantity of transverse reinforcement shall not be less than one-half of that required in the potential plastic hinge region". This requirement has been met on the attached mark-up by providing the same spiral at double the spacing in these areas.
7. For the column at grid F2 level 3 we are left with a zone at mid height where the spiral is governed by the minimum requirements for shear and that is R10 @ 150mm pitch. For the column at grid D2 level 3, because the potential plastic hinge zones are longer, there is only a very small length that would be governed by the minimum shear reinforcement and from a practical point of view the R10 spiral at 80mm pitch would probably be continued through here.
8. In summary, as shown in the attached mark-up, for the indicator column at grid F2 level 3 the required spiral varied from R10@50mm pitch in the hinge and joint zones to R10@150mm pitch in the mid-height zone. For the indicator column at grid D2 level 3 the required spiral varied from R10@40mm pitch in the hinge and joint zones to

R10@80mm pitch in the mid-height zone.

Regards

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From: David Hopkins [mailto:David.Hopkins@dbh.govt.nz]
Sent: Thursday, 2 February 2012 7:08 p.m.
To: Clark Hyland; 'Ashley Smith (StructureSmith)'; 'Sherwyn Williams'; 'Nigel Priestley'; 'Rob Jury'; 'Adam Thornton'; Vicky Newton; 'Dr Helen Anderson'; 'Stefano Pampanin'; 'Peter Fehl'; 'Peter Millar'; 'George Skimming'; 'Marshall Cook'
Cc: Avon Adams; Ann Clark; Kate Ryder; Sarah Morton; Raphael Hilbron; Vicky Newton; Neil Green; Nerys Parry; Mike Stannard; David Kelly
Subject: CTV Presentation Updated to 2 Feb for your information

Dear Clark, Ashley and Panel Members

The attached is the result of revisions made since yesterday responding to feedback from DBH and their advisers. This is basically final, though the slide on Column movement comparison may be revised. I have included it because in the previous slide I will mention that about 20 times the confinement steel is required to meet the standard. (Subject to confirmation by Ashley Smith). There is a danger (almost a certainty) that this will be interpreted as the CTV columns as being 20 times under strength. Hence the new slide which establishes that in spite of the discrepancy in reinforcing, the difference in drift capacity is not nearly as great – less than 2 to 1.

Please review the slides and let me know if you have any comment or concerns. As you know, the presentation is being given on behalf of the Panel and the Consultants. I am getting very helpful feedback from the dry runs we have had but I am conscious that the changes made in response to the feedback are acceptable to you.

In the absence of any comment, I shall proceed on the basis that the attached is acceptable. Thank you for your support.

Regards

David

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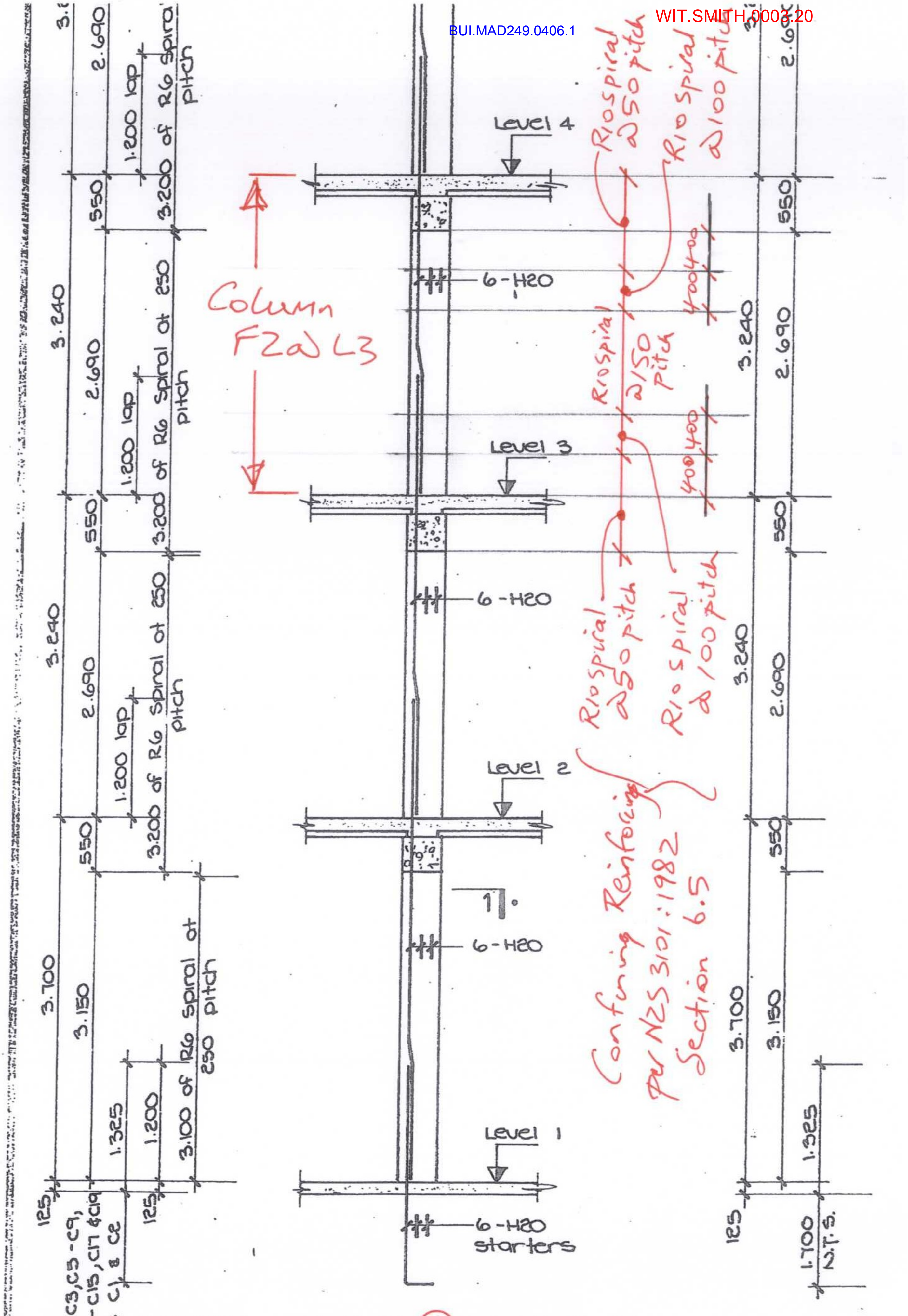
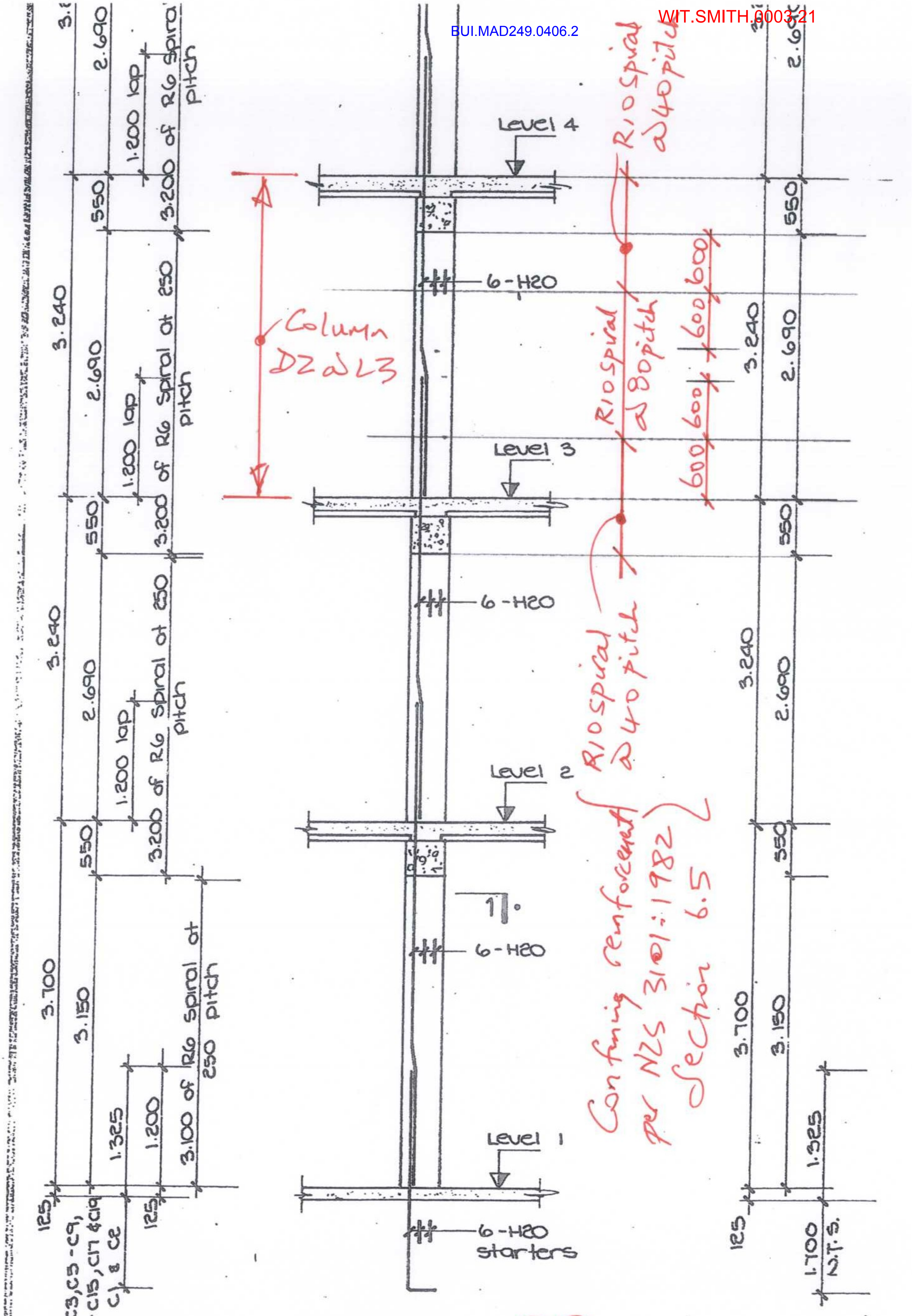
"C"



COLUMNS C1, C2, C3, C5, C6, C7, C8, C9, C11 C12 C13 C14 C15 C17 & C19

AS 3/2/12



COLUMNS C1, C2, C3, C5, C6, C7, C8, C9, C11 C12 C13 C14 C15 C17 & C19

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"D"

EVALUATION OF A 10-STOREY BUILDING USING ALTERNATIVE STRUCTURAL SYSTEMS

by D K Bull*

1. Introduction

This paper will discuss evaluations of several model designs for a 10 storey building, both from a cost aspect and from a structural design point of view. The design, construction and cost benefits of high strength columns will be outlined.

The model building is square in plan, with a floor plan area, to the outside of the cladding panels, of 900 m^2.

Within the building envelope seven structural systems were designed and detailed. The analysis of the various structural systems allowed discussion of the factors influencing design, detailing and choice of structural systems. Costing of each structure included: piling, ground floor and lift pits, frame (columns, beams, floors, structural walls), cladding, fire rating were necessary, and "Preliminary and General".

The structural designs were undertaken by Rankine & Hill Ltd, Wellington with Cement and Concrete Association consulting on the optimisation of High Strength Concrete usage. The quantity surveyors Rider Hunt Holmes Cook Ltd coordinated the measure and obtaining of current market rates for fabricating, supply and erection of the materials. The contractor, Downer and Company Ltd, presented programming and "Preliminary and General" commentary for this study.

2. The Building

2.1 The Model

The 10 storey building was chosen as an indicative model that could lead to establishing trends in design, construction and costs.

Ten storeys was chosen as a "half-way point" for discussion of trends for squat commercial buildings as well as for tall commercial buildings (up to 20 storeys). Discussions with the construction industry prior to commencement of the study suggested that if commercial development was going to occur, in the next two to three years, it would possibly be in the fringe Central Business Districts of major cities or in provincial centres, at around 10 storeys high.

The square 30 m x 30 m building was selected to minimise effort for the design and costing exercises. The plan area was chosen to maximise floor space and to maximise the resulting column loads. In doing so the structural elements were chosen to carry loads efficiently, replicating current design alternatives, and to minimise the "learning curves" for the contractor.

The site chosen for the building was on the fringe of the Auckland Central Business District.

2.2 Design Parameters

The superimposed gravity loads adopted for the general office loads were as follows: Superimposed dead load = 0.5 kPa and live load = 3.5 kPa. However the live load was taken as 2.5 kPa in combinations involving earthquake.

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* D K Bull, Structural Engineer, Cement & Concrete Association of New Zealand

Wind loads were derived from NZS 4203:1984 but were not found to be critical when compared with the seismic loads.

A three dimensional modal analysis was undertaken for each structural system to assess the effects of earthquake, using the ETABS90 program version 5.11. The design spectra and method of NZS 4203:1984 was generally adopted, except scaling of design actions to a level corresponding to a base shear equivalent to 90% of that derived from the equivalent static procedure was not undertaken, in line with the procedure proposed in the draft DZ4203 (January 1991).

The drift limits of NZS 4203:1984 were adopted to ensure control of P-delta effects. Computed drift ratios were scaled by 2.0/SM x Vbase equivalent static/Vbase etabs in compliance with the Code.[2]

The structural design elements were designed to:

NZS 3101:1982	Concrete frames, shear walls, piles, floor systems
NZS 3404:1989 and HERA Design Guide Vol 2	Composite steel beams, columns, EBF's

The building was assumed to be founded on either sandstones and siltstones located approx 5m below ground level or Basalt located 8 m below ground. Piled foundations were considered necessary and the following parameters adopted:

Rupture compressive strength:	Qult =	6.0 MPa
Rupture tensile strength:	Tult =	3.0 MPa
		sidewall friction on rock socket

2.3 The Structures

2.3.1 Type 1: Reinforced Concrete Seismic Peripheral Frame and Interior Gravity Frame

This structure type represents the conventional seismic frame currently being used in New Zealand. Refer to Figure 1, Appendix A.

Four identical exterior frames were used, on each side of the building. A reasonably deep beam was required to control lateral drifts, so a 1200 mm deep beam was chosen. This beam also doubled as an exterior panel. The beams were mid-span insitu spliced. The peripheral column sizes were kept to a minimum (850 mm x 400 mm), however this resulted in a fairly heavy reinforcing requirement (2.8% typical longitudinal steel content).

Inertias of 0.65 Ig and 0.9 Ig for the beams and columns were chosen respectively to represent cracked section stiffness during earthquake loading.

The floor system consisted of 300 mm "Dycore" and 65 mm topping, with units at 2400 mm centres and 105 mm insitu concrete on "Traydec" spanning between units.

Internal beams were half-height precast, acting continuously over interior columns and pin-ended at the perimeter columns.

Within the frame type, two options were considered for the interior gravity frame:

Option 1: Conventional Compressive Strength Concrete Columns

The interior frame columns were designed for 30 MPa and 40 MPa concrete.

Column size (all interior columns):	600 mm x 500 mm
Range of longitudinal steel content	= 1.0% to 2.7%.

Option 2: High Strength Concrete Columns

In the lower storeys of the interior gravity frame High Strength Concrete was used (up to 70 MPa).

Column size (all interior columns):	450 mm x 450 mm
Typical longitudinal steel content	= 0.8%.

2.3.2 Type 2: Interior Shear Wall System with Peripheral Conventional Strength Reinforced Concrete Gravity Frames.

This structure type utilises a conventional strength (30 MPa) concrete throughout.

The shear wall system was coupled walls in one principle direction and simple cantilever walls in the other principle direction. Refer to Figure 2, Appendix A.

The basic layout of the shear walls was dictated by the service core arrangement which remains the same for all five structural types. The core layout was detailed by an international architectural group. The coupling beam size of 1500 was as deep as possible, within the serviceability constraints, maximising wall efficiency.

Inertia values adopted for loadings about both principle axis were 0.6 Ig for walls and 0.4 Ig for the coupling beams.

The floor system was the same as Type 1.

The internal beams were half-height precast built into the shear walls and pin-ended at the perimeter beams.

The cladding assumed for this structural type is a 1200 mm high, 100 mm thick precast spandrel panel with aluminium window infills (the glazing is not included in any of the costings as it is common to all Types).

The four corner columns were:	400 mm x 400 mm
Typical longitudinal steel content	= 1.0% provided (minimum steel was required by design).
The eight mid-face columns were:	550 mm x 400 mm
Range of longitudinal steel content	= 0.9% to 3.0%.

2.3.3 Type 3: Interior Shear Wall System with Peripheral Reinforced High Strength Concrete Gravity Frame.

This structure and cladding is the same as Type 2 except that High Strength Concrete is used in the lower storey mid-face columns of the gravity frame. Refer to Figure 2, Appendix A.

The four corner columns (full height 30 MPa):	400 mm x 400 mm
Typical longitudinal steel content	= 1.0% provided (minimum steel required by design).

The eight mid-face columns were: 400 mm x 400 mm
Longitudinal steel content = 1.0% provided
(minimum steel required by design).

Concrete compressive strengths ranged between 30 MPa and 55 MPa.

2.3.4 Type 4: Interior Shear Wall System with Peripheral Structural Steel Gravity Frame.

This structural type uses the same shear wall system as Types 2 and 3. The average steel content of wall 2 is lower than the others, reflecting the lighter structure of the steel gravity frame. Refer to Figure 3, Appendix A.

The steel frame uses Universal Columns (UC's) for the column sections and Universal Beams (610 UB 101) for the internal primary beams. These beams were propped to limit stresses around penetrations during construction (though a 610 UB101 without penetrations is adequate without propping).

Two options for the floor system were considered:

Option 1: 360 UB 45's and 460 UB 67's were used in the composite secondary beams. These beams were not propped. The flooring was 120 mm, 30 MPa concrete filled "Hi-bond" metal decking (0.75 mm thick).

Option 2: Tribeams 400 TB 45's and 500 TB 50's were used in composite action as secondary beams.

These beams were unpropped. The flooring was "Floorspan 830" metal decking (0.95 mm thick).

2.3.5 Type 5: Structural Steel Eccentrically Braced Frame with Structural Steel Gravity Frames

Four peripheral frames of identical size were used. The column spacing was chosen to have three equal spaces between the columns along each side of the building. Refer Figure 4, Appendix A.

The shear link size of 850 mm was determined from a HERA recommendation that D < e < 1.6 Mp/Vp. Link stiffeners and lateral braces were determined from NZS 3404:1989 Part 2 and HERA Design Guide Vol 2 Chapter 15.

Beam and brace sizes were determined from seismic and gravity forces and section geometry requirements in NZS 3404:1989, Part 2.

Lateral drift control required the EBF columns to be concrete encased to stiffen the frames.

A continuous twin spine beam option was adopted. These beams (2-460 UB 74) were angled from the internal to external columns to eliminate the need for larger exterior beams that would be required to support the spine beams if they had framed into the exterior beams. The spine beam option provided greater flexibility for services with only a moderate steel weight increase. It also allowed the secondary beams to be identical, as the secondary beams pass over the spine beams, thereby eliminating the need for different lengths required to suit the diagonal configuration of the single primary beams.

2.3.6 Structural Steel Composite Beams

Propped and unpropped options were considered. Generally the size increase was small changing from the propped to unpropped option, hence the unpropped option was adopted for the secondary and majority of the primary beams.

For all UB section composite beams, a deflection limit of span/300 under superimposed loading with full composite action was adopted, with long term effects, shrinkage etc all considered. Further, for the unpropped option chosen, a deflection limit of span/250 to a maximum of 30 mm under wet concrete loading during construction was also imposed. A ponding allowance of 15% was used in assessing the concrete weight and a later check found this to be a reasonable estimate.

For the UB beam types, it was found that the "construction" deflection limit generally governed the design of the secondary beams. Precambering of the UB sections was considered in lieu of adopting the deflection limit but it was considered that the savings in steel mass of generally less than 10 kg/m was not sufficient to offset the costs of precambering.

The tribeam section composite beams were sized directly from load span tables, supplied by the manufacturer, Steltech.

2.3.7 Piles

The piles to the building were utilised to take out the lateral seismic base shear of the superstructure. It was assumed that the ground floor slab acts as a diaphragm to distribute lateral loads to all piles.

The seismic base shear was taken as the upper limit corresponding to SM = 2 (based on Vbase, ETABS (unscaled)) in accordance with NZS 4203: 1984. It was found that the base shear associated with the over strength of the seismic system of each structure generally exceeded this value.

The Reese Matlock approach was utilised in the design of the piles and distribution of lateral load.

Bell sizes were determined from the areas required to support axial loads given the fore mentioned founding parameters.

Tension was not found to govern bell sizes.

2.3.8 Fire Rating Requirements

All support structures were detailed to meet a minimum fire rating requirement of 1.5 hours in accordance with NZS 1900, Chapter 5.

3. General Cost Evaluation

3.1 Scheduled Quantities and Rates

The quantity surveyor was appointed by the Cement and Concrete Association to produce the schedule of quantities from the drawings provided and coordinate the obtaining of current market rates for the Auckland-based study.

In order to evaluate regional trends this model building was also costed in Wellington and Christchurch by the quantity surveyor's offices in those centres.

3.2 Contractor's Evaluation

A national contractor was requested to evaluate the structures, as designed, for "Preliminary and General" considerations.

The contractor commented that earlier input on the design phase could lead to savings in structural cost. Every contractor has work practices and equipment specific to their team. Consideration of these specifics as early as possible in the overall design can lead to reduced structural costs.

The contractor emphasised that the "P & G"s applied to this 10 storey, 900 m² floor plan, fringe Auckland, Wellington and Christchurch CBDs. Other heights, size or locations would change the absolute dollar values.

The contractor further stressed that the "P & G" and work content were at rates current as at August

1991. Subsequent movements in competitive material supply or availability of critical trades and equipment could influence these costs.

As an example of trade influence, the steel tendering costs in these evaluations were taken from one company who has been consistently under-bidding all other steel fabricator competitors. The capacity of that company to handle several contracts simultaneously at the same price is questionable and fabrication costs would become more realistically set by other fabricators.

An example of the influence of availability of equipment to a contractor is when a contract can be won because a contractor, who has specialist equipment, such as climbing wall-forms, does not have to carry the cost of buy-in that equipment.

The "Preliminary and General" in the tables and graphs following relate only to the foundations, the structural frame and floors. No allowance has been made for fixing precast panels or fire rating work on the assumption that these operations are not on the critical path.

Fire rating work, along with the cladding, particularly for structural steel, can be critical when the fire-rating material needs to be protected from the weather. Fire-rating, to not be on the critical path, needs to progress, in parallel, with cladding fixing rather than following it.

3.3 Summary of the General Cost Evaluation

As noted earlier, Type 1 frames use the peripheral structural beam as a spandrel. All other types use 100 mm thick precast panel 1200mm high and aluminium window infill as cladding (the glazing is not included in the costing as it is common to all Types).

Figure 5 shows a graph of the Auckland "structure"(including floors, structural frame, and foundations), "P and G" and "cladding". The high strength concrete option for Type 1: seismic frame and interior gravity frame was the most cost effective. As can be seen the "all concrete" structures were less costly than the alternatives incorporating structural steel.

This study showed that the EBF structural steel frame was the most economic steel system, ranked fourth most cost effective behind the "all concrete" structures. An alternative combined steel braced, concrete structure was not part of this initial study, but offers some scope for future analysis.

Figure 6 shows a graph of the Auckland structural systems.

Figures 7 and 8, for Wellington and Figures 9 and 10 for Christchurch costings follow the same trends as Auckland.

One of the interesting conceptions that occurred at the beginning of the study was that shear wall buildings were perceived as being significantly more expensive than the seismic frame equivalents. The study has shown that, even without high performance formwork (conventional shutters on a floor-by-floor basis were assumed), the shear wall structures were of a similar cost to the seismic frames (the Christchurch result indicated that the shear walls were less costly).

The development of fair comparative costs of different structural material systems is difficult particularly in respect of erection times. Fast structural frame erection gives the impression of shorter overall completion but it is building completion for occupation that is the governing concern. Overseas studies show that it is building complexity, shape and contractor experience that dominate the contract building time rather than a choice of steel or concrete structures.

This is confirmed in a recent study undertaken in Australia. The study found that on a number of Melbourne projects, speed of construction appeared to have more to do with the builder chosen and his efficiency of job organisation including planning, industrial relations, resources of labour, materials and plant utilisation than with the choice of structural systems (R Richardson, Rider Hunt Pty Ltd, 1990[6]). The influences of cash flow requirements for concrete and steel buildings have not been included in the cost study to date.

AUCKLAND

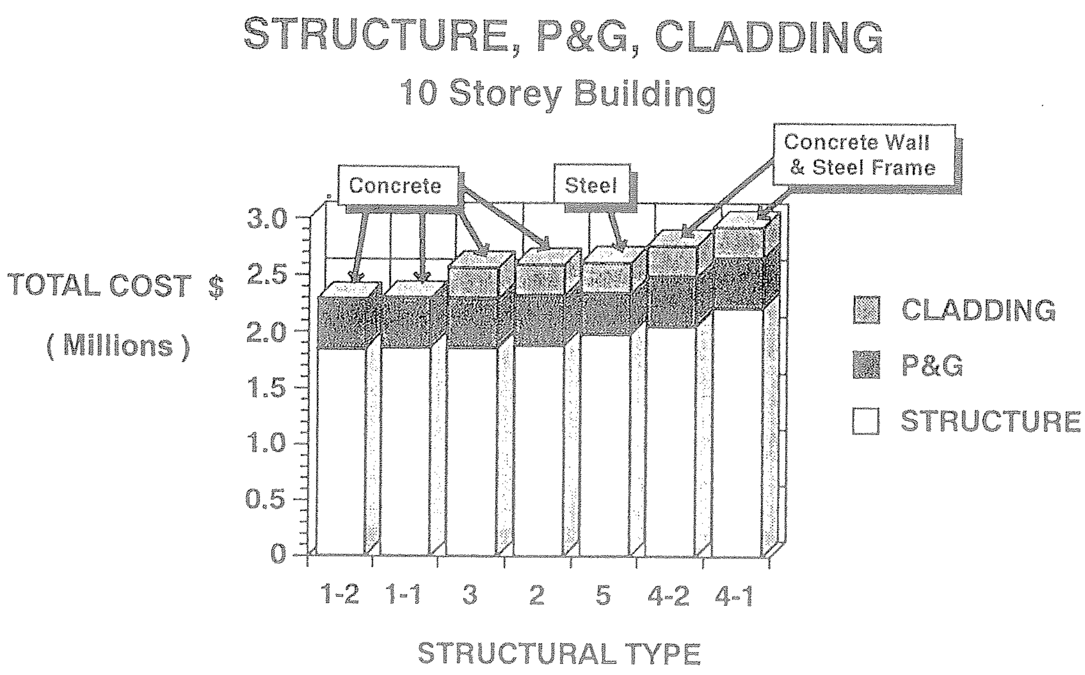


Figure 5 : Comparative Building Estimates : Auckland.

AUCKLAND

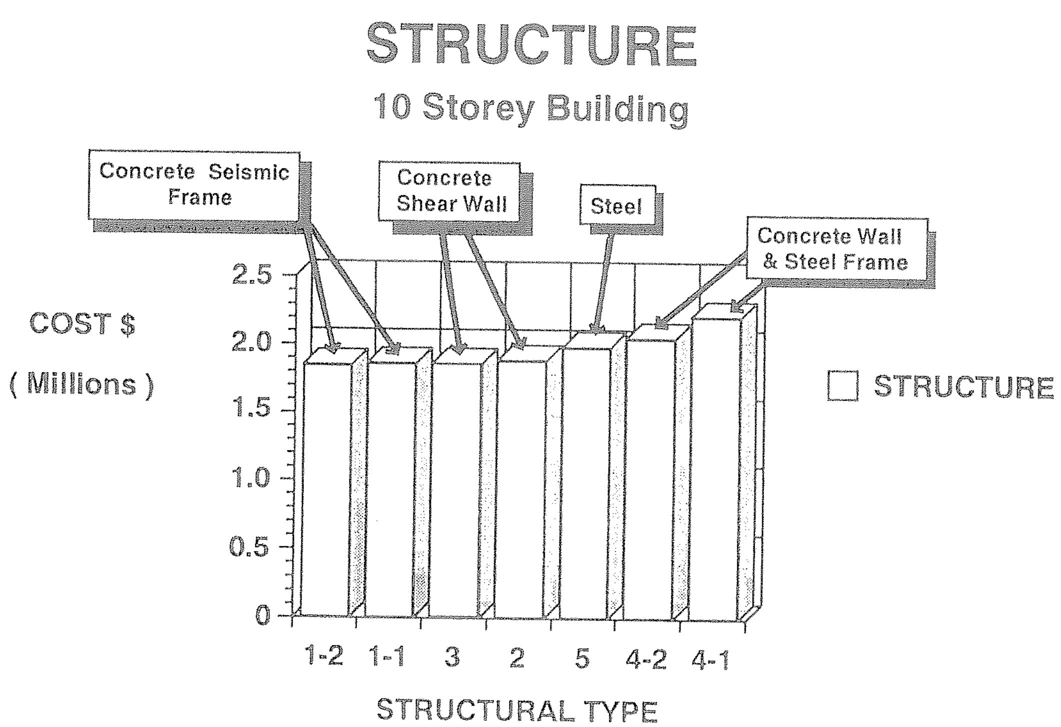


Figure 6 : Comparative Structural System Estimates : Auckland.

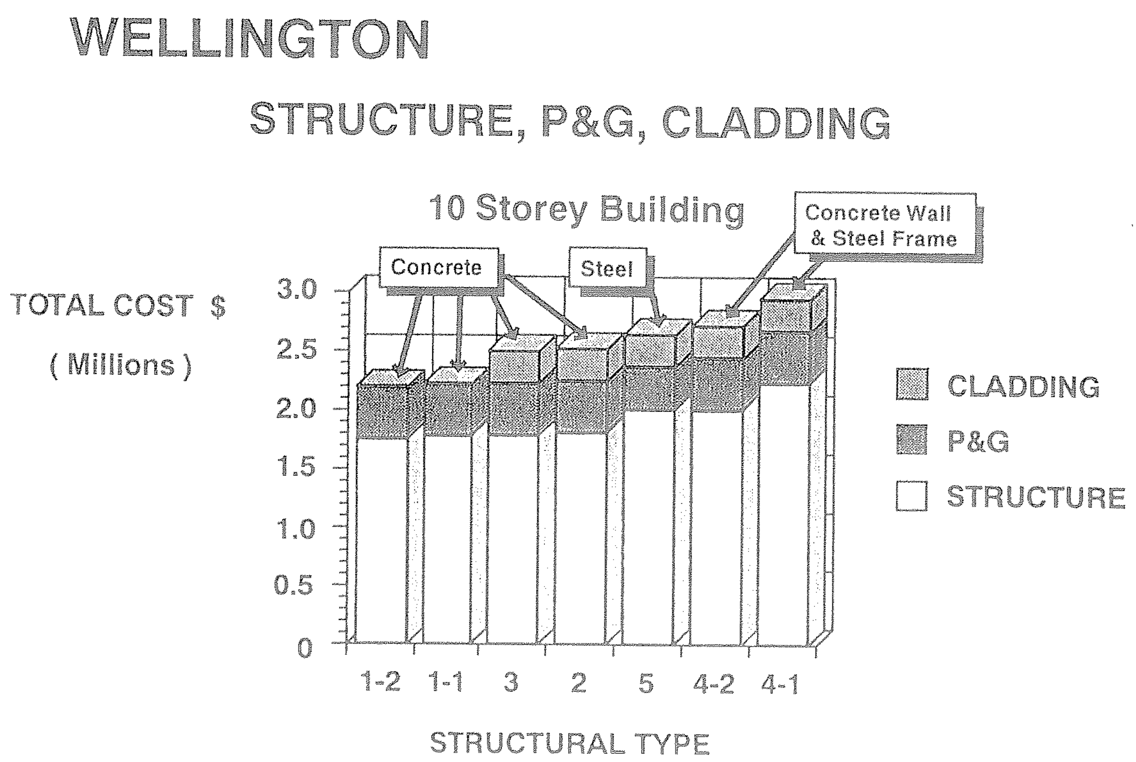


Figure 7 : Comparative Building Estimates : Wellington

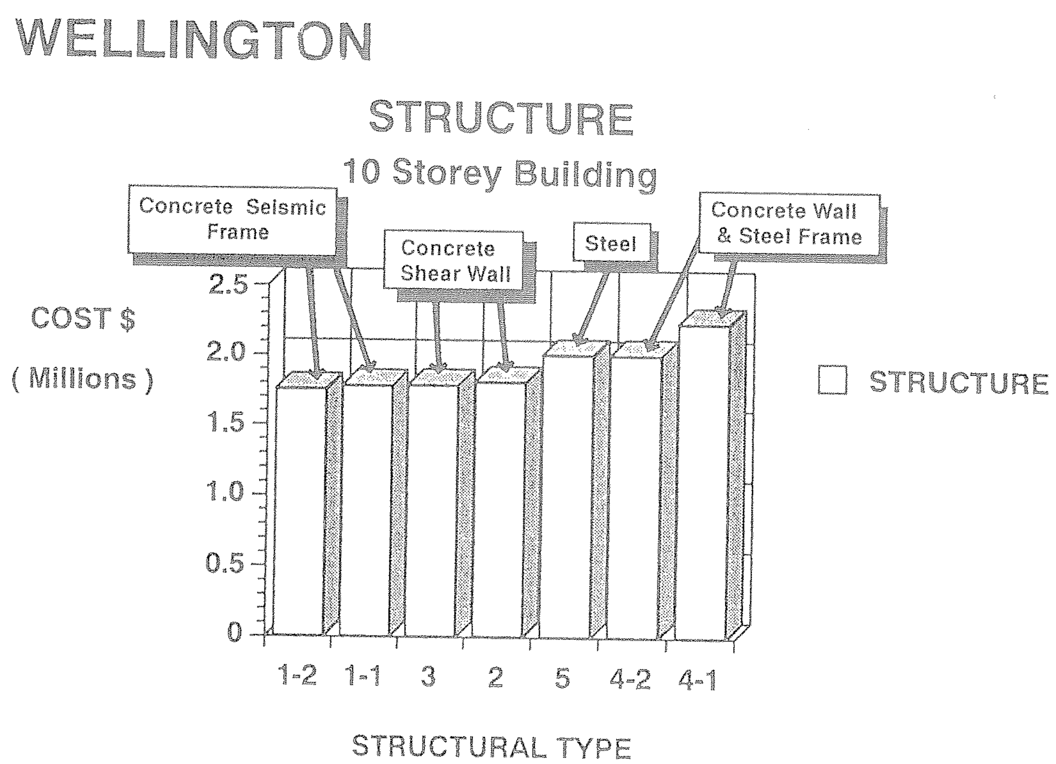


Figure 8 : Comparative Structural System Estimates : Wellington

CHRISTCHURCH

STRUCTURE, P&G, CLADDING

10 Storey Building

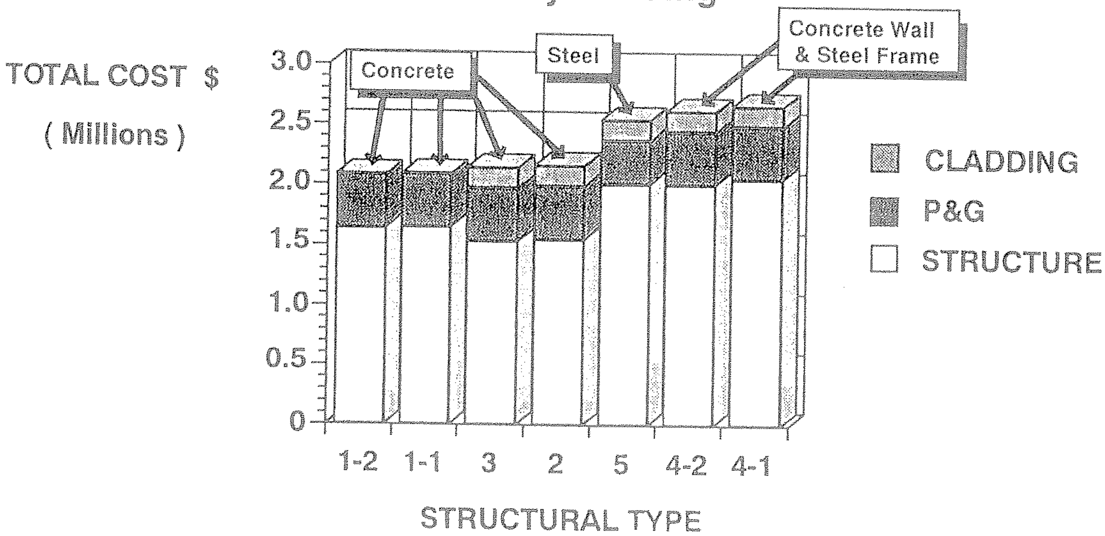


Figure 9 : Comparative Building Estimates : Christchurch.

CHRISTCHURCH

STRUCTURE

10 Storey Building

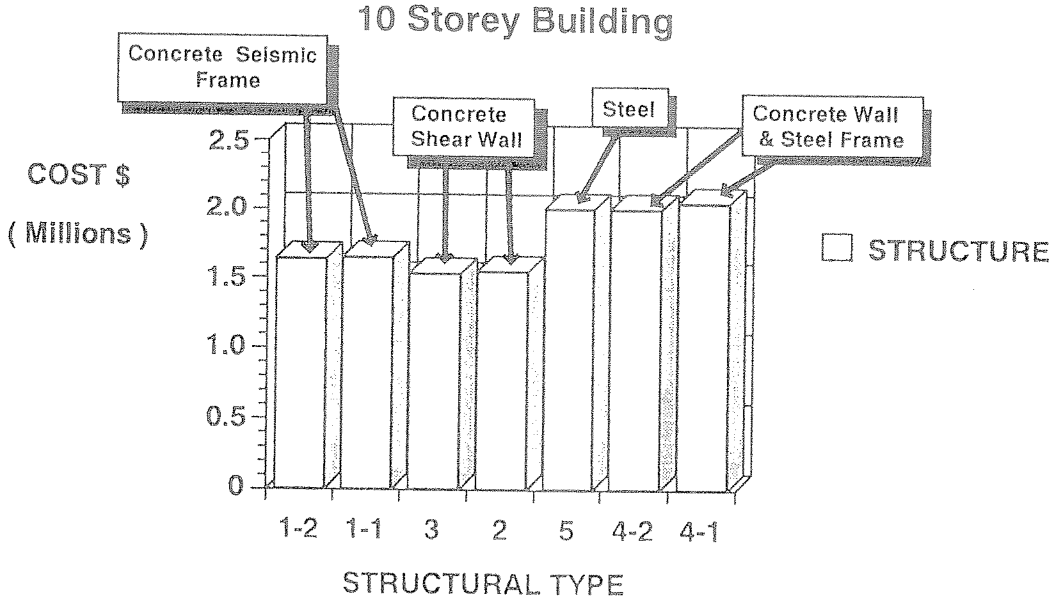


Figure 10 : Comparative Structural System Estimates : Christchurch.

3.3.1 Conclusion of the General Cost Evaluation

The study was undertaken for Auckland where, because of current material costs, one might expect structural steel to be assessed at its most favourable.

This structural model indicates, in generic terms, that structural steel buildings do not have any cost advantages over reinforced concrete types in the Auckland market. The costing analysis for Wellington and Christchurch further enforces that structural steel buildings do not have any cost advantages over structural concrete.

4. Evaluation of High Strength Concrete Columns

4.1 Benefits of Using High Strength Concrete in Columns

The cost efficiencies of carrying axial load using High Strength Concrete are detailed in Section 4.3.

Other benefits that support the choice of High Strength Concrete columns are as follows:

- maximise usable interior space: through smaller columns (potentially less in number)
- enhanced architectural scope.
- standard column sizes and reinforcing cages (reduced weight): which result in minimising learning curves to the contractor, increased uses of the same formwork, decrease in cranage effort
- reduction in differential movement between perimeter columns and cores
- reduction in axial shortening due to lower shrinkage and creep
- increased Modulus of Elasticity
- higher early strength with reductions of deflections under construction loads, and earlier stripping of formwork.

4.2 Designing a High Strength Concrete Column

In determining the size and steel contents of the High Strength Concrete alternatives in the study, certain strategies were used as outlined by Bull and Chisholm [4].

4.2.1 Column Size

The following considerations led to the final solutions:

1. To minimise the section size, minimise formwork costs and reduce concrete volume (see section 4.3), it was decided to start with the maximum concrete compression strength available where special admixtures, such as silica fume, were not needed (approximate 75 to 85 MPa). Such admixtures significantly increase the price of the concrete, however there will be situations where cost-benefit analysis will indicate that higher strengths would be viable.

2. A practical column size was chosen between 350 mm and 450 mm square in order to accommodate either dowels (via Drosbach tubes or sleeve connectors) from a precast column to precast beams or the extension of the column cages through the beam/column joint for either precast or insitu columns.

4.2.2 Column Fabrication

Details can be developed for both precast and insitu column construction. There is considerable potential to move column fabrication "off-site" by precasting. When precast beams are used, the column-to-beam connection is via grouted sleeves or ducts. These specialist grouting operations need to be programmed.

For the study an insitu column was decided upon with an insitu beam/column joint pour (of the same strength f'c as the column below).

Insitu beam/column joint pours require care in marrying the floor topping concrete with the "mush-roomed" high strength concrete that must be placed as part of the floor, around the column location.

Grouting operations of precast units should use grout, on interfaces and in ducting, of 10_MPa higher than that of the poured concrete. Special care is necessary in the joint area of the precast beam if the beam/column joint is cast as part of the beam. This joint zone, which is part of the beam, must have the same strength as the column below.

In the study the beams were made continuous across the columns. In the situation of simply supported beams sitting on column corbels or connected by hanger details to columns, then seismic detailing and design discussed in Section 4.2.4 need not apply. Such assemblies lend themselves very readily to multi-level precast column units.

4.2.3 Longitudinal Steel Content

Minimising steel content will significantly reduce the cost of a column (see section 4.3).

The minimum longitudinal steel content of NZS3101 [1] is 0.8%. Overseas a steel content typically used is 1% for column design using High Strength Concrete. In the study approximately 1% steel content was targeted.

In order to accommodate the varying axial loads on the columns (increasing down the building), with a fixed longitudinal steel content, the f'c for the columns must be varied. (ie_maximum f'c at the base where the load is maximised). In implementing the aim of minimising longitudinal steel content, along the column size considerations of Section 4.2.1, and the maximising of f'c, the solutions typically iterated to a zero steel content. The code requirement of a minimum steel content of 0.8% would then govern.

Column design computer programs allow for rapid iteration when varing concrete compressive strengths are used. In this study the Cement and Concrete Association column design program "CONCOL" Version 3 was used.

4.2.4 Design Philosophies : Gravity Frames

At the inception of the study there was much debate as to whether "gravity" frames should or should not be carrying lateral load? And were the designs to be "limited ductility" approaches or "full ductility" approaches.

4.2.4.1 Gravity Frames - Participation in Carrying Lateral Load:

For a frame to carry lateral load it must have some continuity between columns and beams. In the study, concrete gravity frames had continuity provided. By necessity, in Type 1: seismic frame, the interior beams required continuity over the columns to keep the beam deflections sufficiently low for that depth of beam (beam depth was selected to allow services to pass underneath). As part of the construction detail the tops and bottoms of the columns were built into the beams via an insitu beam/column joint pour, hence frame continuity was achieved.

Type 2 and 3 gravity frames, on two sides of the building, had beams requiring full continuity and the columns were similarly built in.

NZS 3101[1] Clause 3.5.6.10, 3.5.10 and 3.5.14 give guidance here.

Clause 3.5.6.10:

This clause covers interior columns of gravity-load dominated ductile frames. The commentary to this clause elaborates that such frames form a reasonable portion of the lateral load carrying system. These provisions were principally intended for three to four storey ductile frames with larger column spacings.

These provisions were not intended for the design of frames in which deformations are controlled by shear walls or substantial earthquake-dominated external frames. Further these provisions were not intended to apply to frames designed as secondary elements in accordance with clause 3.5.14.

The study has shown that for Type 1: Option 1: seismic frame, the interior (30 MPa) gravity frame attracted less than 10% of the design base shear. It clearly did not represent a structure in terms of clause 3.5.6.10.

In the shear-wall types (Types 2 and 3), the gravity frames took negligible lateral load.

Clause 3.5.10: Structures with Limited Ductility:

This clause applies to primary lateral load structural systems that were designed to limited ductility criteria. Here again the gravity frames of Types 1, 2 and 3 were not of this primary category. These gravity frames were not carrying any significant proportion of primary action.

Clause 3.5.14: Secondary Structural Elements:

This clause goes to great lengths to caution and guide the designer. Only in the situation of very stiff lateral load resisting systems, such as shear walls, the associated gravity frame (which carries little lateral load) would then be classed as a secondary structure. (For examples, Types 2 and 3 structures of the study.)

The commentary recommends caution in assuming levels of participation in carrying lateral load by gravity frames, and that it would be prudent to consider any frame system (gravity dominated or otherwise) to be part of the primary lateral load carrying system. Accordingly such participating gravity frames would be detailed as primary members.

Type 1: Option 1 is a case in point, the interior frame was conservatively described as a secondary structure and hence ignored for lateral load carrying. It may have been prudent and more economical to include the interior frame in order to stiffen up the whole structure, potentially reducing the loading on the external columns and aiding in the control of interstorey drift.

NZS 3101[1] therefore suggests that unless there are special circumstances any structure or group of substructures (gravity frames, walls, etc) should be considered as fully participating members in resisting lateral load.

Note: "fully participating members" can be designed and detailed for either or a combination of "full ductility" or "limited ductility". If "full ductility" is used then the frame is dealt with like any other seismic frame with full ductility detailing and "capacity" design.

4.2.4.2 Gravity Frames: Full Ductility or Limited Ductility?

The first reaction after deciding that the gravity structures were "secondary", was to start designing on the basis of "members not designed for seismic loading"[1] and go to a "limited ductility" approach. This proved to be not completely appropriate.

Clause 3.5.14 and its commentary set out quite definite guidelines to the design and detailing of secondary structures. Section 14: "Seismic Requirements for Structures of Limited Ductility"[1] directs the designer back to Section 3 for clarification.

Clause 3.5.14 defines two types of secondary structure: Group 1 and Group 2.

In short:

- Group 1 elements are detailed through separations, and are not to be subjected to induced loading by full post-elastic primary structure deformations.

- Group 2 elements are subjected to induced loadings from primary structure deformations.

The gravity frames of the study are typically Group 2.

Clause 3.5.14.3[1] sets the criteria for the type of design approach, be it elastic response, limited ductility or fully ductile behaviour.

To assess what approach a secondary structure (Group 2) is to be designed to, the designer needs to have analysed what strength is provided in relation to the full post-elastic primary structure deformations. This is effectively a determination of the ductility demand on the secondary structure.

4.2.4.2.1 Gravity Frames: Secondary Structure (Group 2) Design:

The following approach was used to design the gravity frames of the Type 1, 2 and 3 structures.

1. Design the frame beams and columns for the loading:
   1.4D + 1.7 LR
   taking in account moment redistribution and code[1] minimum eccentricities.

2. Having opted for "secondary structure" status moment magnification in accordance with Clause 6.4.11[1] was necessary. Being "secondary" in nature the gravity frames were considered to be "braced against sidesway".

3. A check design against code[1] design earthquake effects and code[1] minimum eccentricities was done. It proved not critical.

4. The gravity frame now designed under pure gravity was evaluated against elastic response of the frame under going full post-elastic deformation (of the primary structure). In this study it was apparent that: columns hinging top and bottom would occur before half the post-elastic deformation occurred. Under Clause 3.5.14.2(f) the columns had to be detailed for full ductility (confinement, and anti-buckling reinforcement and development).

   The commentary to this clause indicates that limited capacity design for shear in the columns is necessary (based on overstrength column hinging) without dynamic magnification but it was not necessary to amplify column moments for higher order effects. Dimension restrictions of Clause 6.5.2 were also applied. Column hinging in the "braced" gravity frame is not a problem as the overall lateral stability of the structure is taken care of by the primary structures (external frame or shear walls).

5. The gravity frame beams were evaluated against the overstrength actions of the columns. In the study the beams were unlikely to yield in the top steel with a negative moment, however the bottom steel was likely to yield with the positive moment. Additional beam longitudinal steel was provided to cover the sagging moment associated with the column hinging.

   The beams were considered in terms of the secondary structure status. The beams were not sufficiently strong over the column hinging nor did the provided beam strength equate to a fully elastic responding structure, therefore ductility detailing was required (confinement and anti-buckling reinforcement). Ductility detailing was provided in the beams at the column faces. Such transverse reinforcing is prudent and generally not significantly expensive. Columns and beam hinges occurring concurrently in a "braced" gravity frame do not lead to a collapse mechanism. In the study, limited beam hinging did not significantly alter the axial loads on the columns, hence the columns overstrengths were not modified.

6. Beam/column joints would be designed for ductility considerations.

7. Even if a designer chooses not to include the gravity frames in the structural analysis, at some stage, it may be necessary to evaluate the sub-frames of the gravity structure for overall ductility demand. It may therefore be prudent to include the gravity frames in the early stages of the analysis.

Summary

Designing gravity frames as secondary structures can lead to economies in those frames as limited capacity design can be used. Dynamic magnification is not used in column hinges, column shear is amplified by overstrength only; beams need not have longitudinal steel increased to accommodate column overstrengths, providing the appropriate ductility detailing is used in potential plastic hinge zones.

4.2.4.3 Gravity Frames - Negligible Lateral Load Capacity

Sections 4.2.4.1 to 4.2.4.2.1 covered the development of gravity frames that could carry some lateral load.

Details can be developed that virtually reduce the lateral load-carrying ability of the gravity frame to nil.

There are three principle ways of minimising continuity between beams and columns:

1. Simply-supported beams on column corbels.
   The top steel at the column face is usually nominal.

2. Beams connected to columns via hanger details. For example, 45 degrees reinforcing bar hangers act as "pin-ended" details.

3. Pure-propping: Beam sitting on top of a prop-column.
   Where the column is fixed to the beams above and below by very nominal reinforcing or dowels. The dowels or reinforcing may not even be fully anchored.

It is debatable as to whether 0.8% longitudinal reinforcement (minimum steel content) is necessary. With P-delta effects of inter-storey drift some level of shear transfer in the dowel connection is required. Engineer judgement will be necessary to develop an appropriate detail.

4.2.4.3.1 Column Design

With the corbel and hanger details the column design criteria is applied axial load at actual eccentricities plus any P-delta effects (ie no applied continuity moments).

For the prop-column the design criteria is either code[1] minimum eccentricities or P-delta effects. It may be prudent to allow for some construction tolerances along with the P-delta effects.

In the prop-column case, consider that if a column was 3.6 metres tall and that for a maximum allowable inter-storey drift of 0.01 times the inter-storey height the maximum delta is 36 mm. Note that if the axial force was to remain in the middle third of the section depth then the column depth would be six times 36 mm = 216 mm. If the axial load is in the middle third then at no time does the extreme fibre of the column go into tension.

The columns designed to negligible continuity therefore do not contribute to lateral load carrying and can not be expected to form plastic hinges during seismic attack. Therefore detailing to the 'general' and 'non-seismic' sections of NZS 3101[1] should be all that is required.

4.2.4.4 Capacity Reduction Factors for Combined Axial and Flexural Design

The study has shown that in order to reduce longitudinal steel and reduce concrete compressive strength a capacity reduction factor of 0.9 should be used. When 0.9 is used however, Clause 6.4.7[1] requires hoop and ties or spirals of a minimum quantity to extend over the full length of the column.

Analysis shows that the cost of the stirrups between potential plastic hinge zones, now required by Clause 6.4.7 are significantly offset by the savings in longitudinal steel and f'c that would have otherwise been required when using a capacity reduction factor of 0.7 (which does not require confinement steel, in the same magnitude in between plastic hinge zones).

Further there is a stage in the upper part of building, when using 0.9, a minimum longitudinal steel content and a minimum f'c (say 30 MPa) are reached. Slightly above this level, designing with a capacity reduction factor of 0.7 will result in the same minimums and remove the obligation for full height confinement. This will lead to further savings.

4.2.4.5 Concrete Compressive Strengths Above 50 MPa

It has been assumed, for the exercise that NZS 3101 can cater for f'c up to 70 MPa. Though strictly such design is outside the Code[1], current discussions in the industry indicates that the code requirements may be applicable in this f'c range.

This is an important area of future research and assimilation of overseas technology for New Zealand application. Addressing this area prior to the up and coming review of NZS 3101[1] will be required.

4.3 Cost Evaluation: High Strength vs Conventional Concrete

The study, for a particular column, demonstrated that High Strength Concrete, in the lower storeys, produce a more cost-effective solution.

As indicated above, designing gravity frames as "secondary structures", in accordance with Clause 3.5.14[1], can lead to economies in reinforcement.

Tables 1 and 2 demonstrate the cost savings in using High Strength Concrete in the lower storeys.

As mentioned above Code [1] requirements were applied to f'c above 50 MPa. Current research by Li Bing, Park and Tanaka [8] shows that NZS3101 [1] confinement requirements are likely to require amendment for the ductile behaviour expectations. Therefore the reported savings in transverse steel of Tables 1 and 2, based on the current NZS3101, may be reduced since with higher f'c's there may be an associated increase in transverse steel volumes.

Nevertheless it is anticipated that there will still remain a significant overall saving from reduced concrete, longitudinal steel and formwork costs.

4.3.1 High Strength Concrete Columns: Cost Benefits and Trends

Tables 1 and 2 are specific to the 10 storey, 900 m² floor plan structures and the options described within each structural type.

TYPE: CONCRETE: COLUMN SIZE:	Type 1: Option 1 Conventional 600x500	Type 1: Option 2 HS Concrete 450x450	SAVINGS using HS Concrete
Concrete	$8,400	$6,660	$1,740
Reinforcement:			
Longitudinal	$7,120	$3,210	$3,910
Transverse	$13,360	$9,340	$4,020
Formwork	$23,760	$19,440	$4,320
		* nett SAVINGS:	$13,990

* for 4 - 10 storey columns.

Table 1: Cost Comparisons of 10 storeys of Interior Gravity Columns (4 of)

TYPE: CONCRETE: COLUMN SIZE:	Type 2 Conventional 550x400	Type 3 HS Concrete 400x400	SAVINGS using HS Concrete
Concrete	$12,320	$9,300	$3,020
Reinforcement:			
Longitudinal	$17,450	$6,090	$11,360
Transverse	$16,980	$15,460	$1,520
Formwork	$37,650	$31,680	$5,970
		* nett SAVINGS:	$21,870

* for 8 - 10 storey columns.

Table 2: Cost Comparisons of 10 Storeys of Peripheral Gravity Columns (8 mid-face columns)

However trends from these specific models are of a general nature and are applicable to multi-storey gravity frame design:

1. Concrete Compressive Strength f'c Increases:
   - there is a reduction in overall cost of a column, because: longitudinal steel reduces (as more axial load is carried by the concrete in lieu of reinforcing steel)
   - cross-sectional area reduces (architectural, fit-out, and tenant benefits)
   - formwork reduces (material, labour and cranage reductions)

- building height may increase
  (for a given cost or structural configuration high strength concrete allows an increased height of structure. In the United States Munn[5] referenced that high strength concrete was a principle reason for increases in building heights).
- transverse steel reduces
  Note that transverse steel, amongst a number of criteria, is a function of the confinement equations of NZS3101 Clause 6.4.7 and 6.5.4.3. There could be a geometry of section, along an increasing f'c, that increases the transverse steel requirement. This study and others (Australia, Japan, North America) indicate that in practical terms of ensuring the appropriate maintenance of load carrying ability and enhanced ductility that the usual situation is a reduction of transverse steel.

However as previously discussed there are likely to be changes to the clauses of NZS 3101 relating to confinement when using a higher strength concrete. This may well mean that overall reductions in the volumes of transverse steel will not occur.

Result: Using higher f'c's will reduce the overall cost of a column.

2. Interior frames maximise the cost benefits:

As can be seen for a column of 10 storeys the greater accumulative axial load (interior accumulates proportionally more load than an exterior column) the higher the concrete compressive strength f'c is required, in conjunction with minimising of longitudinal steel and column section size. Hence, overall, on a per column basis the greater nett saving is for interior columns.

This trend continues as axial load increases through a combination of increasing building height, tributary floor areas, and imposed loads.

5. General Conclusions

High Strength Concrete is a technically and economically viable tool to enhance the performance and usage of a commercial structure.

The study shows that concrete structures, both conventional and high strength can produce the most cost-effective solutions for low to medium size commercial buildings. As building height increases the benefits of High Strength Concrete further increase therefore the study's conclusions on concrete's cost-effectiveness can be readily extended to taller structures.

Given New Zealand's history of innovative concrete design and construction of reinforced concrete buildings there would appear to be merit in applying existing resources to further improve cycle times and further development of proven concrete building technology. These applications are sounder than pursuing more speculative benefits from alternative systems.

---

6. References

1. "Code of Practise for the Design of Concrete Structures, NZS 3101 Part 1: 1982" And "Commentary on Design of Concrete Structures, NZS 3101 Part 2:1982" and Amendments(1989), Standards Association of New Zealand.

2. "Code of Practise for General Structural Design and Design Loadings for Buildings, NZS 4203:1984", Standards Association of New Zealand.

3. "Steel Structures Code, Part 1:1989: New Zealand Amendments to AS 1250:1981" and "Steel Structures Code, Part 2: 1989, Means of Compliance", Standards Association of New Zealand.

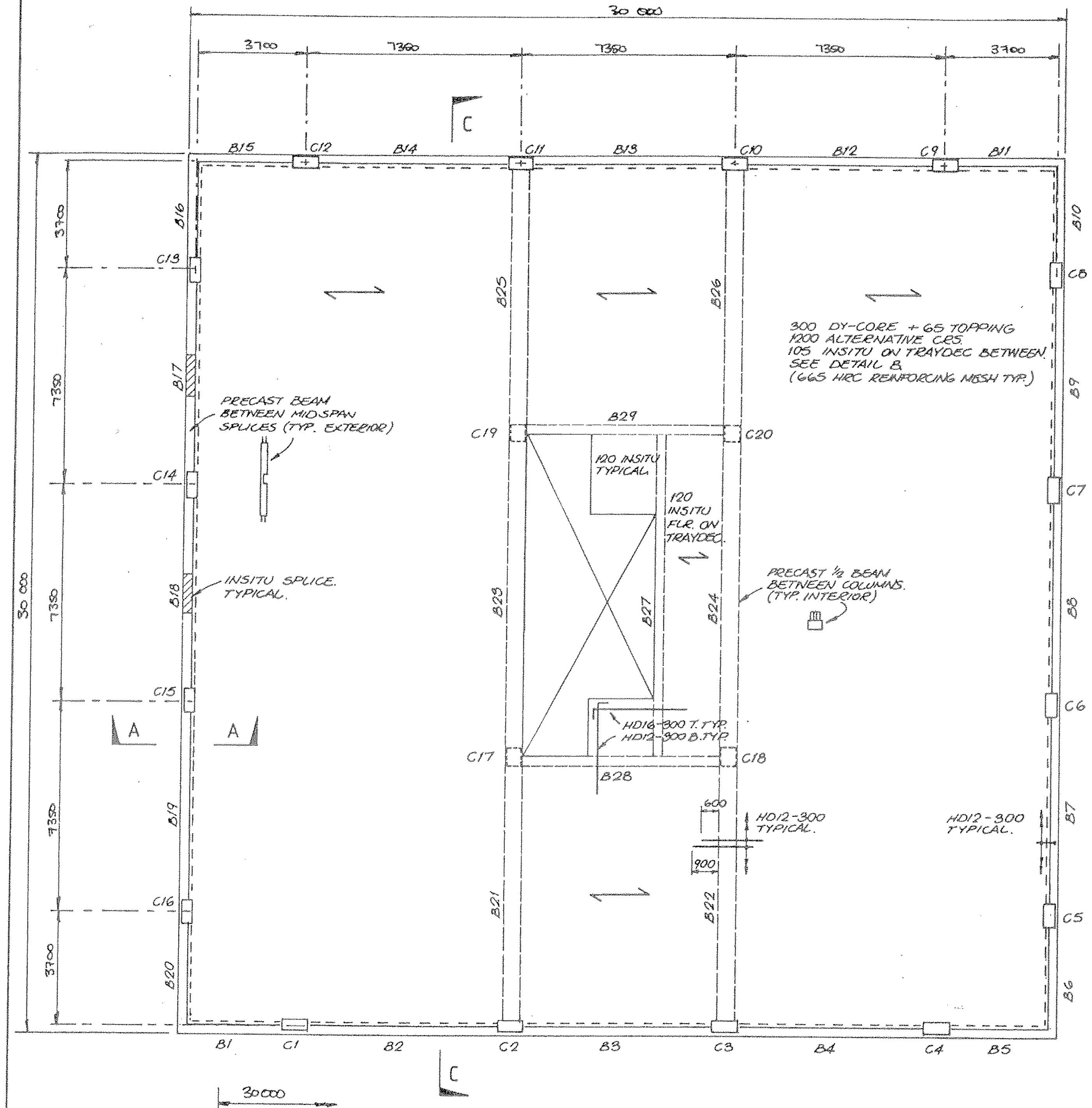
4. Bull, D K, and Chisholm, D H, "High Strength Concrete: Current Overseas Technology and its Applications in New Zealand", Proceedings at the Annual Conference, Institution of Professional Engineers New Zealand, Feb 1991, Auckland.

5. Munn, R L, "Australian High-Strength Concrete", Concrete Institute of Australia News, Vol 16 NZ, June 1990.

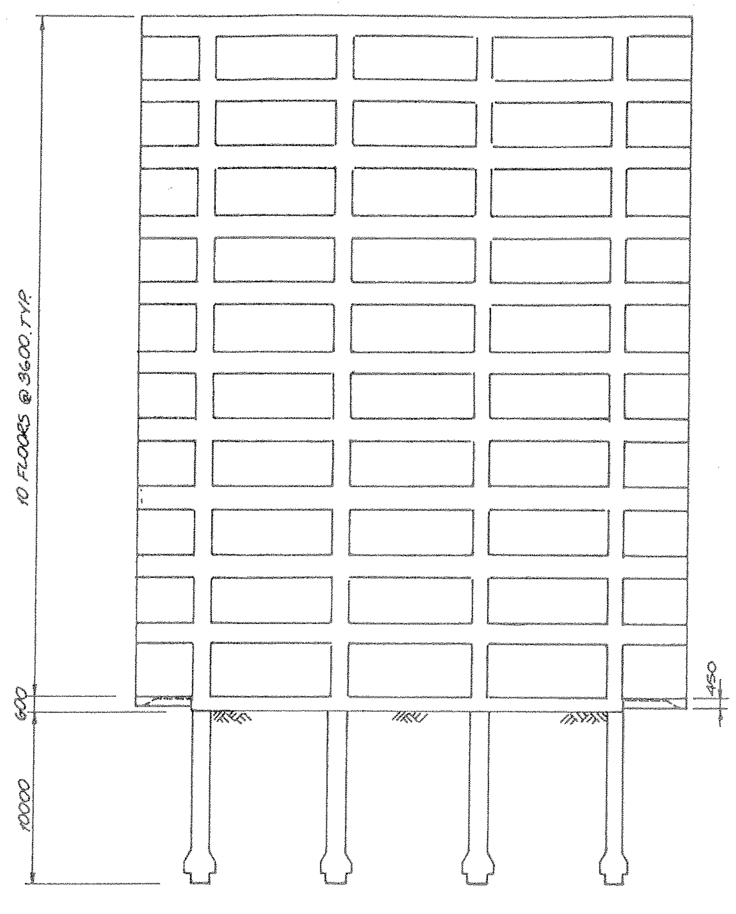
6. Richardson, R, "Case Study: Costing the Alternatives", Proceedings of the Institute for International Research Conference "Concrete..... Steel", March 1990, Sydney, Australia.

7. "New Zealand Structural Steelwork Design Guides: Volume 2", Sept 1989, NZ Heavy Engineering Research Association.

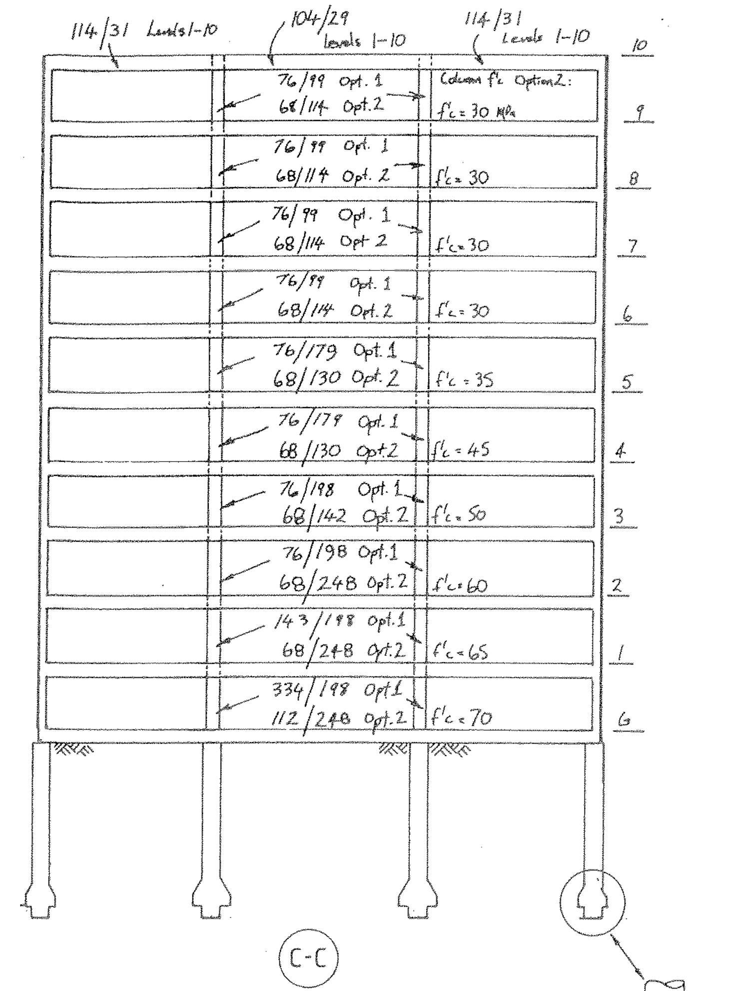
8. Li Bing, Park R and Tanaka H, "Effect of Confinement on the Behaviour of High Strength Concrete columns under Seismic Loading", Proceedings of the Pacific Conference on Earthquake Engineering, NZNSEE, November 1991, Auckland.



TYPICAL FLOOR PLAN

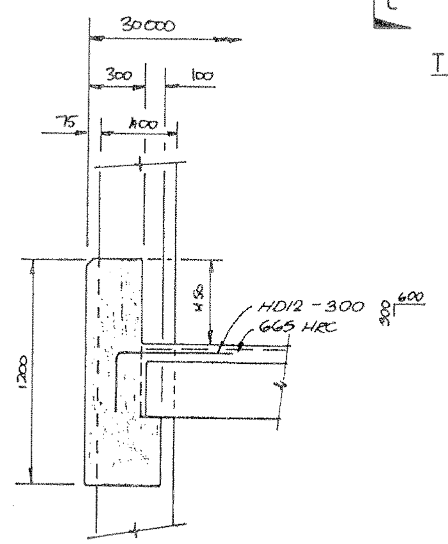


TYPICAL ELEVATION EXTERIOR FRAME



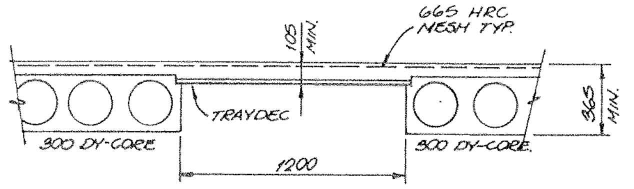
TYPICAL SECTION INTERIOR FRAME

MEMBER TYPE	SIZE	STEEL CONTENT (LONGITUDINAL/TRANSVERSE) kg/m³
COLUMNS ALL LEVELS		
C1 - C16 (f'c = 30)	850 × 400	290/110
C17 - C20	600 × 500 (OPTION 1) 450 × 450 (OPTION 2)	f'c = 30 SEE ELEVATIONS; f'c = varies : see above
BEAMS - GROUND (f'c = 30)		
B2 - B4, B7 - B9, B12 - B14, B17, B19	600 × 600	60/30
BEAMS - LEVELS 1-10 (f'c = 30)		
B1 - B20	1200 × 300/400	78/72
B21 - B26	750 × 600	See ELEVATION C-C
B27	750 × 400	85/14
B28 - B29	550 × 350	100/15
PILES (f'c = 25)		
EXTERIOR	1000 ø (1500 BELL)	135/35
INTERIOR	1000 ø (1800 BELL)	50/35
FLOORING (f'c = 30)		
GROUND	150 INSITU ON GRADE	663 HRC + HD12 - 300 STRS, + 2 HD24 DRAG BARS TO EXTERIOR COLUMNS
LEVELS 1-10	AS NOTED	



A-A

TYPICAL BEAM-COL. FLOOR DETAIL



DETAIL B

NOTES:
1. OPTION 1 - STANDARD REINFORCED CONCRETE STRUCTURE.
OPTION 2 - HIGH STRENGTH CONCRETE INTERIOR COLUMNS

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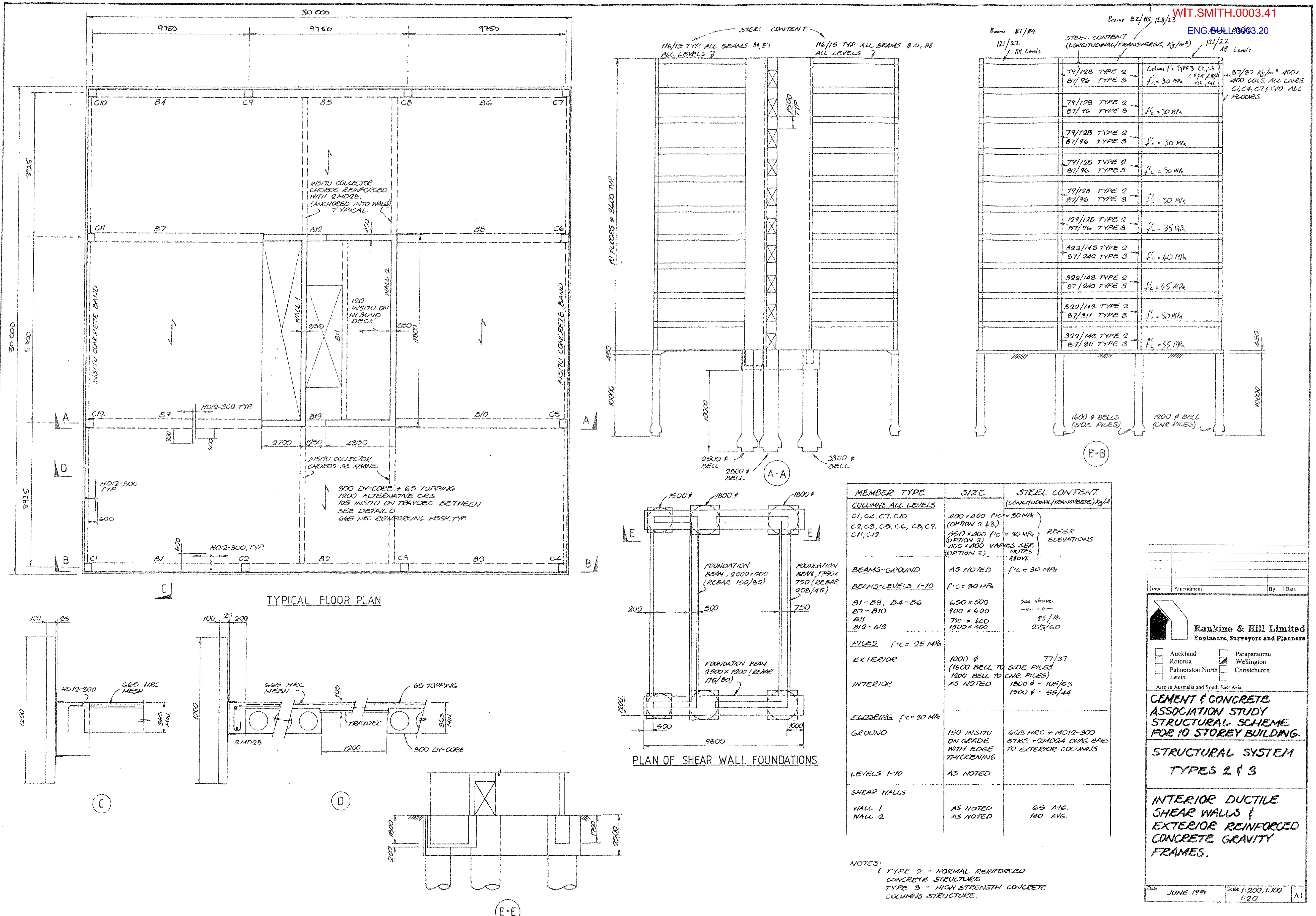
CEMENT & CONCRETE ASSOCIATION STUDY. STRUCTURAL SCHEME FOR 10 STOREY BUILDING.

STRUCTURAL SYSTEM TYPE 1.

DUCTILE MOMENT RESISTING REINFORCED CONCRETE FRAMES & INTERIOR REINFORCED CONCRETE GRAVITY FRAMES.

Date JUNE 1991 | Scale 1:200, 1:100, 1:20 | A1

Figure 1: Structural System Type 1



MEMBER TYPE	SIZE	STEEL CONTENT (LONGITUDINAL/TRANSVERSE) Kg/m³
COLUMNS ALL LEVELS		
C1, C4, C7, C10	400 x 400 f'c = 30 MPa (OPTION 2 & 3)	REFER ELEVATIONS
C2, C3, C5, C6, C8, C9, C11, C12	550 x 400 f'c = 30 MPa (OPTION 2) 400 x 400 VARIES SEE NOTES ABOVE (OPTION 3)	
BEAMS-GROUND	AS NOTED	f'c = 30 MPa
BEAMS-LEVELS 1-10	f'c = 30 MPa	
B1-B3, B4-B6	650 x 500	see above
B7-B10	900 x 600	-"- -"-
B11	750 x 400	85/14
B12-B13	1500 x 400	275/60
PILES f'c = 25 MPa		
EXTERIOR	1000 ∅ (1600 BELL TO SIDE PILES 1200 BELL TO CNR. PILES)	77/37
INTERIOR	AS NOTED	1800 ∅ - 105/53 1500 ∅ - 55/44
FLOORING f'c = 30 MPa		
GROUND	150 INSITU ON GRADE WITH EDGE THICKENING	663 HRC + MD12-300 STRS. + 2MD24 DRAG BARS TO EXTERIOR COLUMNS
LEVELS 1-10	AS NOTED	
SHEAR WALLS		
WALL 1	AS NOTED	65 AVG.
WALL 2	AS NOTED	140 AVG.

NOTES:
1. TYPE 2 - NORMAL REINFORCED CONCRETE STRUCTURE
TYPE 3 - HIGH STRENGTH CONCRETE COLUMNS STRUCTURE.

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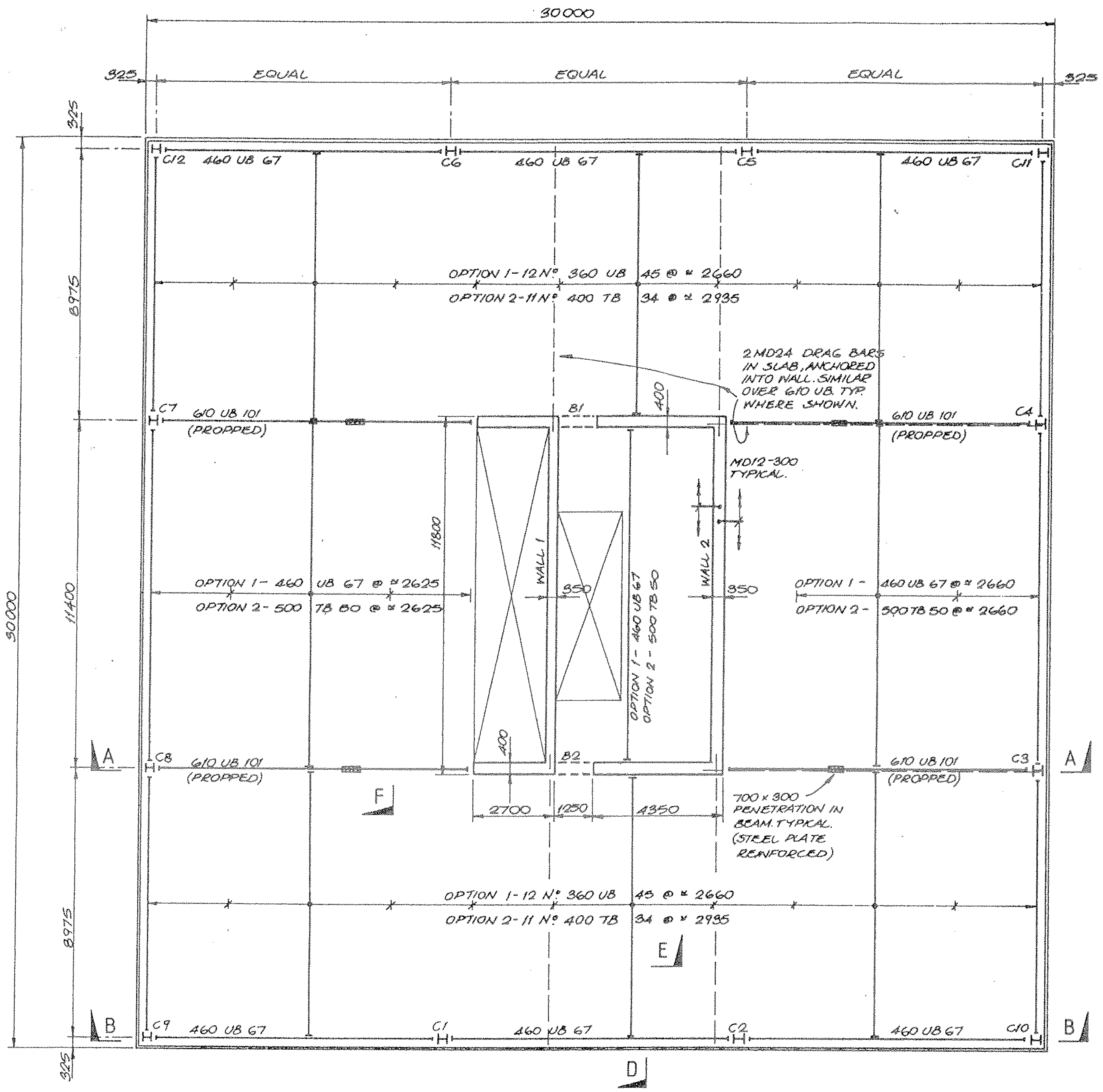
CEMENT & CONCRETE ASSOCIATION STUDY STRUCTURAL SCHEME FOR 10 STOREY BUILDING.

STRUCTURAL SYSTEM TYPES 2 & 3

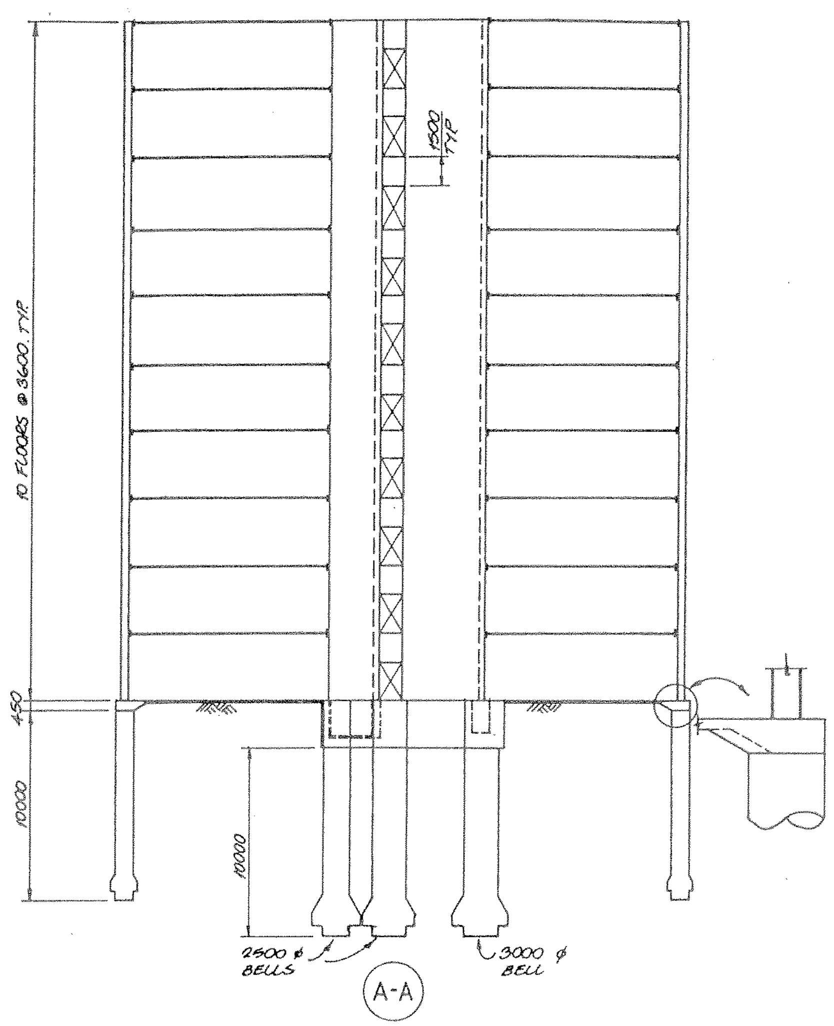
INTERIOR DUCTILE SHEAR WALLS & EXTERIOR REINFORCED CONCRETE GRAVITY FRAMES.

Date JUNE 1991 | Scale 1:200, 1:100, 1:20 | A1

Figure 2: Structural System Type 2 and 3

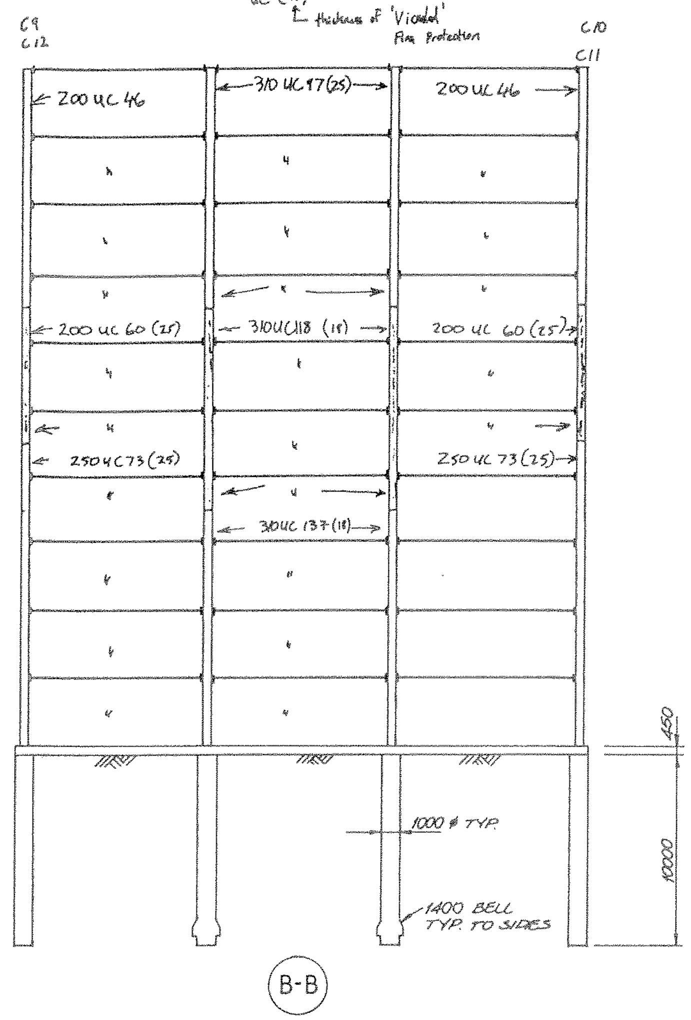


TYPICAL FLOOR PLAN

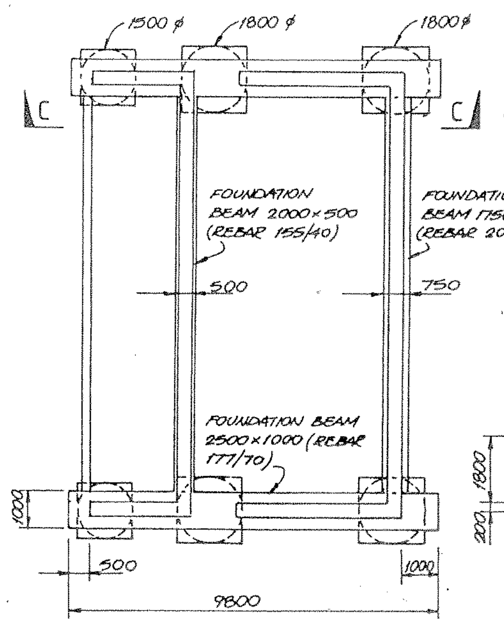


A-A

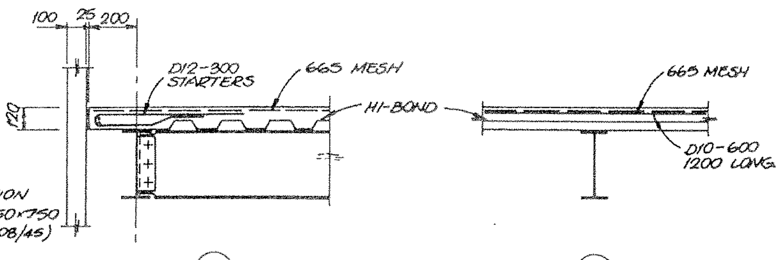
Key: UC (##) thickness of 'Vicuclad' Fire Protection



B-B

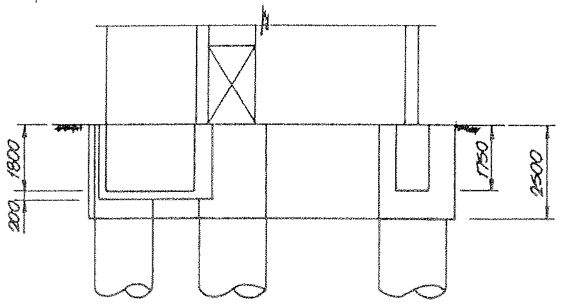


PLAN OF SHEAR WALL FOUNDATIONS



D

E

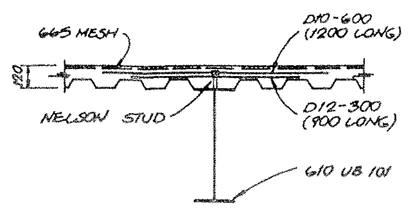


C-C

MEMBER TYPE	SIZE	STEEL CONTENT (LONGITUDINAL/TRANSVERSE) kg/m³
COLUMNS ALL LEVELS		
C1 - C8		REFER ELEVATIONS
C9 - C12		
BEAMS - GROUND	AS NOTED	f'c = 30 MPa
BEAMS - LEVELS 1-10	f'c = 30 MPa	
B1, B2	1500 x 400	190/60
PILES f'c = 25 MPa		
EXTERIOR	1000 ∅ (1400 ∅ BELL C1-C8 ONLY)	60/34
INTERIOR	AS NOTED	1500 ∅ - 52/44; 1800 ∅ 69/46
FLOORING f'c = 30 MPa		
GROUND	150 INSITU ON GRADE WITH EDGE THICKENING	663 HRC + MD12-300 STRS + 2 MD24 DRAG BARS TO EXTERIOR COLUMNS.
LEVELS 1-10 OPT. 1	120 mm WITH HI-BOND METAL DECKING (0.75 mm)	665 HRC + D10-600 T (1200 LONG) OVER ALL BEAMS + D12-300 SLAB EDGE STRS + D12-300 B (900 LONG) OVER 610 UB PRIMARY BEAMS.
OPT. 2	120 mm WITH FLOORSPAN 850 METAL DECKING (0.95 mm)	
STUDS TO BEAMS		
ALL STUDS 19 mm ∅ 100 mm LONG.		
SECONDARY BEAMS	1 STUD EVERY ALTERNATIVE TROUGH; 2 STUDS EVERY OTHER TROUGH	
EXT. PRIMARY BEAMS (460 UB 67)	1 STUD EVERY 220 mm	
INT. PRIMARY BEAMS (610 UB 101)	1 STUD EVERY 175 mm	
SHEAR WALLS f'c = 30 MPa		
WALL 1	AS SHOWN	65 AVG.
WALL 2	AS SHOWN	115 AVG.

NOTE: "FIRE PROTECTION TO STEELWORK"
SPRAY ALL BEAMS WITH ROBERTS M34 MINERAL FIBRE, AS AVAILABLE FROM CROW REFRACTORY INSTALLATIONS NZ LTD TO THE FOLLOWING THICKNESSES.

BEAM	THICKNESS (mm)
360 UB 45	24
460 UB 67	22
610 UB 101	19
400 TB 34	38
500 TB 50	25



F

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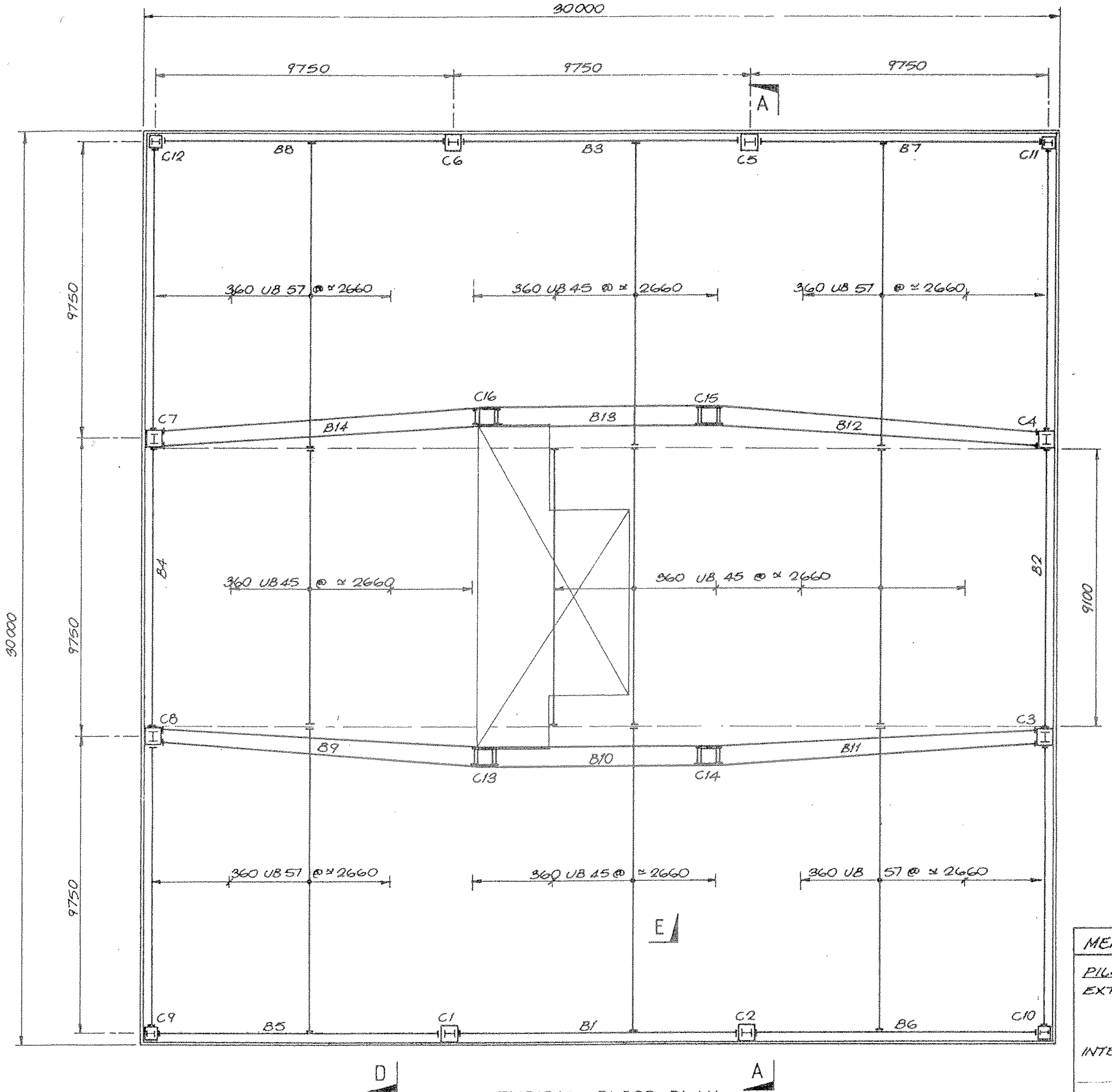
STRUCTURAL SYSTEM TYPE 4.

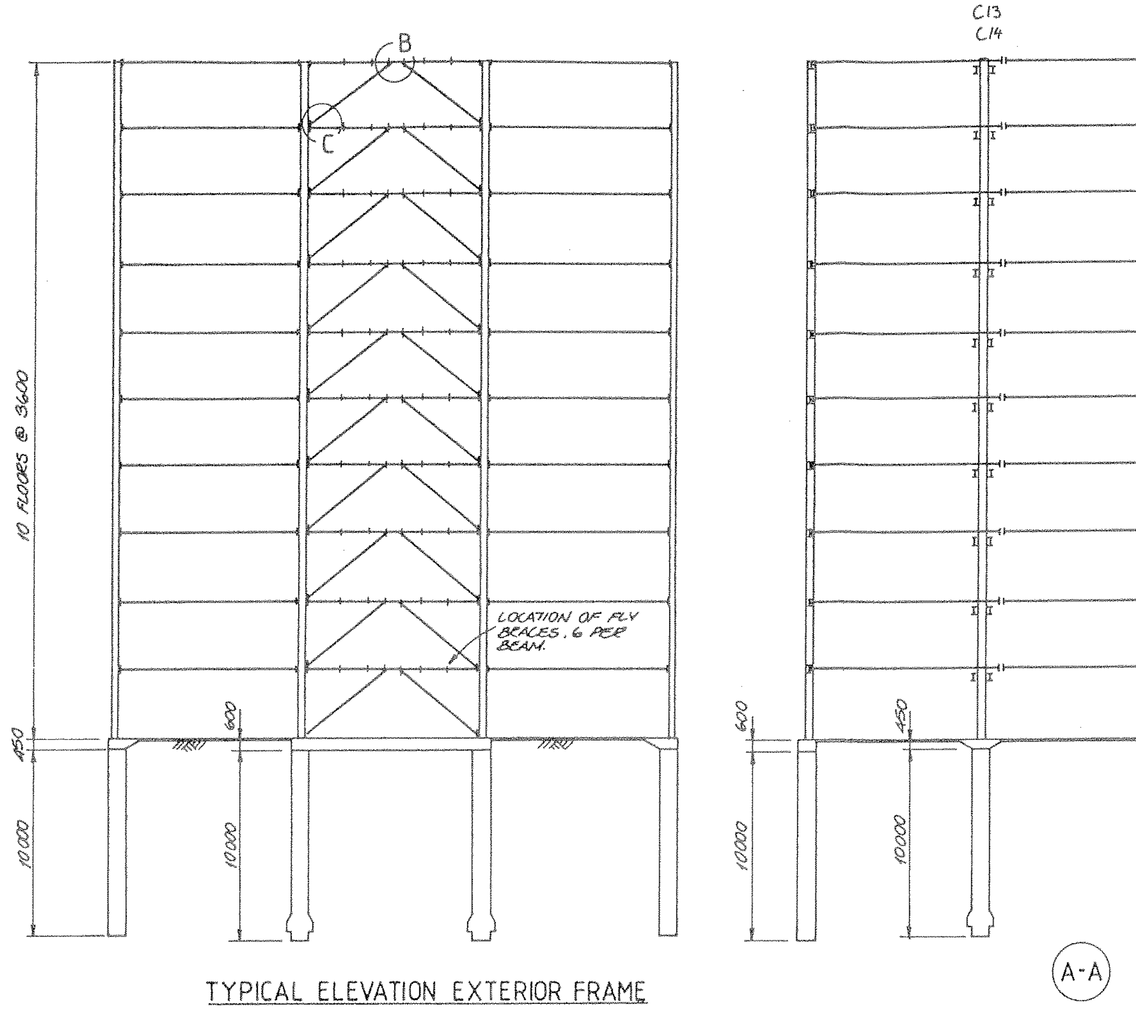
INTERIOR DUCTILE REINFORCED CONCRETE SHEARWALLS + PRIMARY/SECONDARY STEEL COMPOSITE GRAVITY FRAMES

Date: JUNE 1991 | Scale: 1:200, 1:100, 1:20 | A1

Figure 3: Structural System Type 4

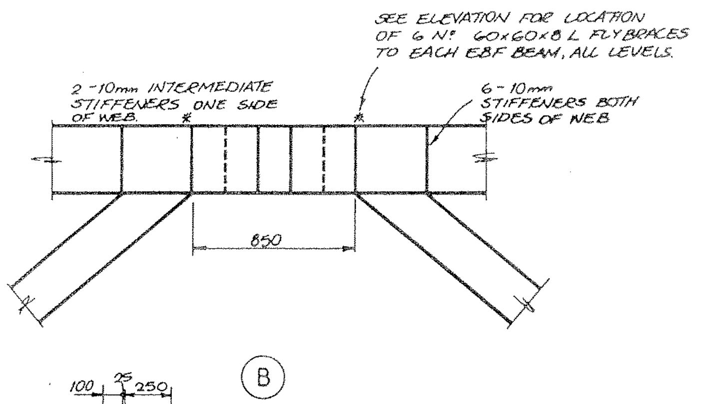


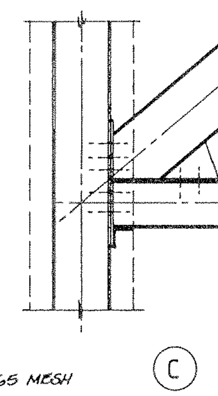
TYPICAL FLOOR PLAN

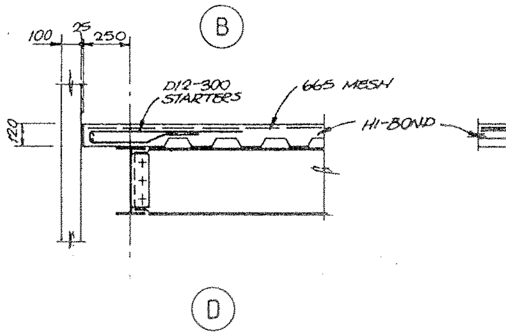


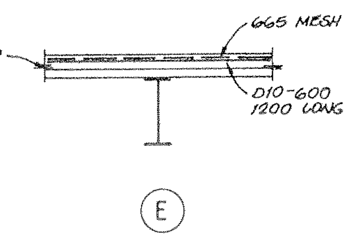
TYPICAL ELEVATION EXTERIOR FRAME

SEE ELEVATION FOR LOCATION OF 6 N° 60x60x8 L FLYBRACES TO EACH EBF BEAM, ALL LEVELS.









MEMBER TYPE	SIZE	STEEL CONTENT (LONGITUDINAL/TRANSVERSE) Kg/m³
PILES f'c = 25 MPa		
EXTERIOR	1000 ø (1500 BELL C1-C8 ONLY)	186/55
INTERIOR	1000 ø (1500 BELL)	50/36
FLOORING f'c = 30 MPa		
GROUND	150 INSITU ON GRADE WITH EDGE THICKENING.	663 HRC + MD12-300 STRS. + 2 MD24 DRAG BARS TO EXTERIOR COLS.
LEVELS 1-10	120mm WITH HI-BOND METAL DECKING (0.75mm)	665 HRC + D10-600 (1200 LONG) OVER ALL BEAMS, + D12-300 SLAB EDGE STRS.

NOTE: "FIRE PROTECTION TO STEELWORK"
SPRAY ALL BEAMS WITH ROBERTS M34 MINERAL FIBRE, AS AVAILABLE FROM CROW REFRACTORY INSTALLATIONS NZ LTD TO THE FOLLOWING THICKNESSES.

BEAM	THICKNESS (mm)
310 UB 40	24
360 UB 45	24
360 UB 57	21
410 UB 54	24
460 UB 67	22
460 UB 74	21
200 UC 60 BRACE	32
250 UC 89 BRACE	30

MEMBER TYPE	SIZE	STEEL CONTENT (LONGITUDINAL/TRANSVERSE) Kg/m³
COLUMNS ALL LEVELS	f'c = 30 MPa	
C1 – C8 Grade 250	550 x 550 + 310 UC 137 (310 UC 118 LEVEL 7-10)	LEVELS GRD.-1 78/117 LEVELS 1-7 78/76 LEVELS 7-10 78/40
C9 – C12 Grade 350	250 UC 73 200 UC 60 200 UC 46	LEVEL GRD-4.5 + LEVEL 4.5-6.5 + LEVEL 6.5-Roof + } 25mm 'Vicuclad'
C13 – C16 Grade 350	450 x 450 + 310 UC 137 310 UC 137 * 310 UC 97	LEVELS GRD-4.5 69/72 LEVELS 4.5-7.5 + 18mm 'Vicuclad' * LEVELS 7.5-Roof + 25mm 'Vicuclad'
BEAMS – GROUND f'c = 30 MPa		
B1 – B4	600 x 600	60/80
BEAMS – LEVELS 1-10		
PRIMARY		
B1 – B4 , LEVELS 1 & 2	410 UB 54	
, LEVELS 3-6	360 UB 45	
, LEVELS 7-10	310 UB 40	
B5 – B8	460 UB 67	
B9 – B14 (PARALLEL SPINE BEAMS)	2 N° 460 UB 74	
SECONDARY	AS NOTED	
BRACES		
LEVELS 1-6	250 UC 89	
LEVELS 7-10	200 UC 60	
STUDS TO BEAMS		
ALL STUDS 19mm ø x 100mm LONG.		
SECONDARY BEAMS	1 STUD EVERY ALTERNATIVE TROUGH 2 STUDS EVERY OTHER TROUGH	
B1 – B4	1 STUD EVERY 300mm (EXCLUDE OVER ACTIVE LINK)	
B5 – B8	1 STUD EVERY 220 mm	

* Alternative 310 UC 97 Encased 69/72

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STRUCTURAL SYSTEM TYPE 5.

STRUCTURAL STEEL ECCENTRICALLY BRACED FRAMES + PRIMARY/ SECONDARY STEEL COMPOSITE GRAVITY FRAMES

Date JUNE 1991 | Scale 1:200, 1:100, 1:20 | A1

Figure 4: Structural System Type 5

"E"

FOREWORD

This revision of NZS 3101 has been written with the objective of producing a concrete design standard which is:

(a) In limit state format;

(b) Compatible with the Loadings Standard, NZS 4203:1992;

(c) Compatible with the New Zealand Building Code in order to enable it to be called up as a Verification Method to the Code.

During the revision process, the opportunity has been taken to incorporate various technical advancements and improvements that have developed since NZS 3101 was first published in 1982.

Minor changes have also been made to the Standard to facilitate future harmonization with the Australian Concrete Structures Code. In particular, new sections covering the design for durability and fire have been based on the corresponding sections of AS 3600, modified as appropriate for New Zealand conditions, materials and regulations. The other non-seismic sections of this Standard are still based largely on the provisions of the building code of the American Concrete Institute, with some of the new provisions of ACI 318-89 being incorporated.

Organizational structure of this Standard

This Standard features an organizational structure which is essentially the same as for NZS 3101:1982. However, for the majority of sections which contain seismic provisions, there is no longer a separate clause covering requirements for members/structures not designed for seismic forces. Such requirements are now included in clause X.3, *General principles and requirements for design*, with seismic provisions being addressed in clause X.4, *Additional design requirements for earthquake effects*. However, sections 7 and 8 maintain separate subsections (7.4, 8.4) for members not governed by seismic actions, which reflects the number of separate design provisions which apply to such members.

Section 3 *Limit state design requirements* and Section 4 *General design requirements*, act as the central framework of the Standard from which the subsequent sections are supported. These sections establish the relationship of this Standard with the Loadings Standard, NZS 4203, and to the Transit New Zealand Bridge Manual.

Accordingly, for efficient use of this Standard, all other sections should be read in conjunction with sections 3 and 4.

A comprehensive commentary is published with the Standard, and it is strongly recommended that the two documents be read together. Commentary clauses are not mandatory. The Appendix to Commentary section 9 provides detailed guidance for designers in specific areas where mandatory provisions are not considered appropriate.

Summary of key technical changes from NZS 3101:1982

The key technical changes from NZS 3101:1982 are briefly summarized below on a section-by-section basis:

Section 3: Limit state design requirements and material properties

- Strength reduction factors have been reduced to maintain the target values of the safety index within the ranges given in the Foreword to NZS 4203:1992.

- The use of higher values for the lower characteristic yield strengths for steel reinforcement and specified compressive cylinder strengths for concrete in design equations is permitted, noting that there are additional specific limitations in the subsequent sections on flexure, shear and bond.

Section 4: General design requirements

- The values of structural ductility factor, μ, contained in this Standard are the same as those of NZS 4203:1992, except that μ shall not be taken as greater than 6 for any concrete structure without special study. For squat ductile cantilever walls, μ is a function of the aspect ratio of the wall, and may be less than 5.

- The structural performance factor, S_p, from NZS 4203 is to be taken as 0.67.

- A new clause addressing the design of precast systems for structural integrity has been added. This clause specifies minimum seating lengths for precast floor elements, along with requirements for ties between precast wall and floor elements to provide a minimum level of robustness.

Section 7: Reinforcement – Details, anchorage and development

- The equations for calculating bond and anchorage have been revised into a more straightforward format.

- The limits on the size of beam reinforcing bars allowed to pass through beam-column joints of ductile frames have been relaxed in line with recent research findings.

Section 8: Flexure with or without axial load

- Modified parameters of the concrete equivalent compressive stress block are provided to enable design for flexure where f'_c is greater than 55 MPa. The minimum permitted tension steel ratio in beams is also increased when f'_c is greater than 30 MPa.

- The seismic provisions for the amount of transverse reinforcement required for concrete confinement in potential plastic hinge regions of columns in ductile frames are made more dependent on the level of axial load. This results in less confining steel in lightly loaded columns and more confining steel in heavily loaded columns than required by NZS 3101:1982.

- The previous provisions for columns which permitted a lower transverse reinforcement content with a reduced strength reduction factor have been deleted. The 1994 Northridge earthquake in particular has highlighted the vulnerability of columns that have low levels of transverse reinforcement over part or all of their length.

Section 9: Shear and torsion

- The use of strut-and-tie models for the design of deep beams, corbels and brackets is recognized and encouraged.

Section 11: Beam - Column joints

- The quantity of shear reinforcement required in joint cores of ductile frames is significantly lower than that required by NZS 3101:1982.

- The equations used to determine horizontal and vertical joint reinforcement are much simpler than those of NZS 3101:1982.

Section 12: Walls

- The limiting thickness of walls to prevent instability in potential plastic hinge regions is made a function of aspect ratio, reinforcement content and structural ductility factor. In heavily reinforced walls this may result in reduced minimum thicknesses compared with the provisions of NZS 3101:1982.

Section 13: Diaphragms

- The requirements for diaphragms have been separated from the section on walls, and expanded upon.

Section 17: Seismic requirements for elements of limited ductility

- This section has been rewritten to reflect an approach which is consistent with the other sections of the Standard. For conventional beam or wall base hinging elements, the provisions embody capacity design principles, but with relaxed detailing provisions in comparison with fully ductile elements.

REVIEW OF STANDARDS

Suggestions for improvement of this Standard will be welcomed. They should be sent to the Chief Executive.

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