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**

STATEMENT OF EVIDENCE OF JOHN HENRY IN RELATION TO THE CTV BUILDING COLLAPSE

DATE OF HEARING: COMMENCING 25 JUNE 2012

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Personal details

1. My full name is John Malcolm Terrence Henry. I am a resident of Christchurch. I am an Associate and Project Leader (Structural) with Eliot Sinclair in its Christchurch office. I have a BE (Hons) from the University of Canterbury. I am a Chartered Professional Engineer.

Background

2. I was contacted by Counsel Assisting the Royal Commission and asked to meet to discuss my knowledge of issues the Royal Commission was examining in relation to the collapse of the CTV building. Following a meeting with counsel I was advised that I would be required to provide evidence and I would be receiving a summons. I have subsequently been served with a summons.

Purpose of evidence

3. The evidence I have been asked to provide to the Royal Commission relates principally to the following matters:

   a. My period of employment by Alan Reay Consulting Engineer from mid 1984 to mid 1985. While I was not employed there at the time the CTV building was designed, and had no involvement in it, I did the structural calculations for the 8 storey Landsborough House during the period I worked for Alan Reay. I have been told by Mr David Harding, and this has also been confirmed by Counsel Assisting, that he did the calculations for the CTV building and the design features of that building were modelled on Landsborough House. I have also been advised by Counsel Assisting that David Harding says he was given a copy of the structural calculations for Landsborough House to use as a template for the method of how to design a building using the ETABS system.

b. The calculations done by David Harding for the CTV building. I have been asked to review these and comment on where and how they differ from the Landsborough House calculations and the significance of these differences. I have also been asked to give evidence on the ETABS system and its use in the design of the CTV building;

c. The design principles used for multi storey shear wall buildings designed in the 1980's and the relevant structural detailing used in their construction;

d. The way the Christchurch City Council (**the Council**) building consent process worked during my time there and, in particular, the different roles played by Bryan Bluck and Graeme Tapper in the consents process and my knowledge of how Alan Reay personally, and Alan Reay Consultants Limited, dealt with the building consent process and with Bryan Bluck and Graeme Tapper.

Code of conduct for expert witnesses

4. Although most of my evidence involves matters of fact, because some of my evidence involves matters of opinion I have been shown the Code of Conduct for expert witnesses. I have been advised that expert witnesses giving evidence to the Royal Commission are required to comply with this Code.

5. I confirm that I have read the Code and I agree to comply with it. I believe that the opinion evidence I give is all within my area of expertise.

Professional engineering experience

6. From 1972 to 1975 I trained as a structural draughtsman with architects and engineers Griffith Moffat & Partners in Christchurch. During that period I attended Christchurch Polytechnic to gain NZCE Civil in 1975.

7. In 1976 I went overseas and worked as a structural draughtsman detailing industrial buildings, first in Durban for structural engineers Horne and Glasson and then in Johannesburg for LSC Brunette, a Zimbabwean based firm.

8. In 1977 I returned to New Zealand to carry on my structural engineering studies. I attended the University of Canterbury from 1978-1979 and gained a Bachelor of Engineering degree with First Class Honours. I continued to work at Griffiths Moffat and Partners as a trainee engineer between studies.

9. I studied structural design under both Professor Park and Professor Paulay, who were regarded as leading structural engineers with international reputations. I will explain the relevance of the relationship I developed with Professor Paulay later in my evidence.

10. I also studied the structural dynamics of buildings under Dr Athol Carr where I learnt the fundamental principles of dynamic analysis for buildings as part of the third professional year structural course.

Holmes Wood Poole and Johnstone: 1980 – 1984

11. After graduating from the University of Canterbury I was employed by Holmes Wood Poole & Johnstone, which for clarity I will refer to as (**Holmes Wood**). This is the predecessor firm of Holmes Consulting Group (**Holmes**).

12. During this time I worked on many significant building projects. I was also trained in the use of the ETABS programme for designing multi-storey buildings. Prior to this time all of the buildings I had worked on were designed by hand methods, as commercial computers or software that could carry out design work were not invented or available.

13. I became a Registered Engineer in 1982 and a Member of the Institute of Professional Engineers in 1983.

Alan Reay Consultants Ltd (ARCL): 1984-1985

14. In 1984 I left Holmes Wood to join Dr Alan Reay's firm which was then called Alan Reay Consulting Engineer. Dr Reay is now the principal of Alan Reay Consultants Limited (**ARCL**), which I will refer to as ARCL. I will refer to the two firms by this name as I am not aware of anything relevant to my evidence that is affected by which of the Alan Reay firms I am referring to.

15. I had been well mentored at Holmes Wood, which had enabled me to become a Registered Engineer in 1982. However, while the experience I received at Holmes Wood was outstanding, I felt that it was time for a change and I was attracted to an advertisement from ARCL for a structural engineer which I thought would give me the opportunity for further advancement. Alan Reay indicated to me during my interview that there was the possibility of a future partnership in the firm.

Lovell-Smith Cusiel: 1985-1986

16. In 1985 I left ARCL and went to work for Dick Cusiel to help analyse the United Building Society building in High Street. This is now the Holiday Inn building. Dick Cusiel offered me part of his office space, which enabled me to also work on my own projects. During this time I became a member of the Association of Consulting Engineers New Zealand.

Holmes Wood/HCG: 1986-1991

17. I worked from the Lovell-Smith Cusiel office for about 18 months before Brian Wood from Holmes Wood asked me come back to help with the Parkroyal Hotel design, which I did. This offered further career advancement. I worked there primarily as the Engineering Manager until 1990 when the firm restructured as Holmes Consulting Group in 1991.

18. During those 4 years, first at Holmes Wood and then at Holmes Consulting Group, I worked on several large building projects. This included the Parkroyal Hotel, the Price Waterhouse Centre, the Antarctic Centre and the strengthening and base isolation of Parliament Buildings and the General Assembly Library.

Own practice: John Henry Consulting: 1991-1992

19. After I left Holmes I established my own practice. During this period I worked mainly on the Port Hills Gondola Building, but also on a number of small residential projects.

Christchurch City Council: 1992-1995

20. In 1992 I joined the Christchurch City Council Building Department. Initially I helped to work out how the Council building control processes could transition from the Local Government Act to the new Building Act. This took about a year. During that time, and for a while afterwards, I worked alongside Graeme Tapper and Bryan Bluck reviewing engineering design building consent applications. Later, after they retired, I took up the role of Building Control Engineer and Building Consents Manager.

Montgomery Watson Harza (MWH): 1996-2002

21. After leaving the Council I worked at MWH as senior engineer and group leader from 1996 to 2002. This involved structural and civil engineering projects.

Eliot Sinclair: 2003 to date

22. I became a Chartered Professional Engineer in 2003. Since 2003 I have worked for Eliot Sinclair where I am employed as an Associate to the firm, carrying out a wide range of civil and structural engineering projects in Christchurch.

Experience of multi-level shear core building design

23. I now address in more detail my structural design experience in the period leading up to, and during which, the CTV building was designed. This section of my evidence also

deals with my knowledge of structural engineering practice for Christchurch multi-storey buildings during this period.

Holmes Wood Poole & Johnstone: 1980 to 1984

24. Before I joined Holmes Wood the firm had designed a number of shear core buildings in Christchurch during the 1970's. They had built up a considerable experience and knowledge base to work from. These included 58 and 64 Kilmore Street (each 4 and 6 storeys) which form part of the ECan Complex and the Spicer House building next door at 329 Durham (5 storeys).

25. All three of these buildings had a seismic structure consisting of either central shear walls or a shear core off-set to one side, adjoining a long shear wall running at right angles along the outside of the building. In the design of these buildings, in order to resist torsional effects, Holmes Wood had taken steps to counteract the eccentric nature of the shear walls by placing additional walls either on the opposite sides or on the ends of the building.

26. I have been carrying out assessments and reporting on earthquake damage to these three buildings over the past year and in the course of doing so I have reviewed the structural drawings and their design. I now see these designs as part of a progression in the developing expertise of Holmes Wood, into which I was introduced in 1980 when I joined the firm. I have seen the extent of structural damage to these buildings and how they performed in terms of the principles and assumptions used in their design.

27. All three of these buildings were designed using the same underlying structural premise for the design of the seismic structure in relation to the gravity load carrying elements. This is the same premise I am now aware the CTV building used, as did the other buildings I will refer to in my evidence. The principle is that the shear walls are designed to carry all the seismic load of the building, leaving the columns and beams to carry only the gravity loads, or building weight, imposed on them. The design of the

gravity load system is therefore simplified because the reinforcing requirements are a lot less compared with those required for seismic loading. The premise underlying this design method is that the stiffness, or inflexibility, of the shear walls prevents the relatively flexible columns and beams from excessive deformation under seismic loading and thereby from suffering significant seismic forces and stresses. However, with this premise, if excessive deflections do occur in a major seismic event the gravity load system is vulnerable to overloading and possible collapse.

28. This is a critical design principle, which recurs throughout my evidence. For ease of description I will refer to it as the "shear wall protected gravity load system".

29. In this type of system the beams typically run in one direction along the building, sometimes, but not always, with an enclosing beam around the perimeter of the building. The joints between the beams and columns are simply reinforced with minimal reinforcing, very much less than if seismic loadings were included in their design. Column reinforcing is also simple and minimal, usually 10mm ties at 250 or 300mm centres.

30. It has been helpful and educational for me to see that although the three eccentric Holmes Wood buildings I have mentioned have suffered damage and cumulative degradation to the point that they are not economical to repair, their shear walls have performed reasonably as expected to provide the assumed protection to the gravity load system, with moderate cracking damage to some of the beam column joints and/or spalling of cover concrete within some end regions of the columns. The beam column joints in these buildings had similar detailing to the CTV building in that the bottom reinforcing bars were turned upwards and terminated in the joint, with light enclosing ties. The column ties were R10 at 250mm centres.

31. During my time with Holmes Wood I overlapped with Andrew (Andy) Buchanan, now Professor Buchanan at the University of Canterbury Engineering School. He was an

Associate of the firm and I worked under his supervision on the Canterbury Savings Bank building, which is now known as the Westpac Centre, at the corner of High and Cashel Streets. This was a 13 storey building with a poured concrete shear core in the centre of the building that was designed and built between 1980 and 1983. The design was done in 1980 and there was a two year construction period. The structure was symmetrical and did not involve any significant eccentricities. In that respect the analysis and design was simplified.

32. This was my first experience of a shear core building utilising the shear wall protected gravity load system. My role involved carrying out the computer analysis of the shear core structure, using a software programme called ETABS, on the University of Canterbury main frame computer facilities. This was the only computer available at the time capable of doing the job.

33. ETABS is a programme that I believe was developed at the University of California at Berkeley in the 1970's, for the purpose of analysing multi-storey buildings in three dimensions taking into account the higher modes of vibration which influence the response of a structure under seismic loading. The programme enables the designer to determine the building response, deflections and member forces with a level of accuracy that was impossible using hand calculation methods.

34. The early versions of ETABS did have limitations modelling wall elements, which was achieved using panel elements joined together with "'virtual" frames. The results, like any computer work, were only as good as the input data, which included assumptions that needed to be borne in mind during the design process. For example, the properties of the beam, column and panel elements needed to be assessed in terms of the degree to which they would crack under earthquake loading. The degree of cracking selected and assigned to some elements could in some cases significantly

affect the deflection results. This sort of sensitivity could be tested using the ETABS model.

35. ETABS was not user friendly in those days. All data was input using punched cards that were processed at the University of Canterbury. The information was supplied for punching, hand written on sheets with rigid formatting. A single digit in the wrong place could cause the analysis to fail midway or could produce rubbish output. Once the model did run successfully, some basic checks were carried out to make sure that the model was running sensibly.

36. These checks included checking the sum of the shears at the base of the building against the total input load, checking that the building period of vibrations were as expected and checking the deflections for any irregularities that would signal any misbehaviour.

37. Once the Westpac building was analysed I carried out the design of the shear core under the mentorship of Andy Buchanan and Russell Poole, which involved Andy sitting with me for about an hour each day to go through the steps involved.

38. In those days a 13 storey building was a major project in Christchurch, utilising a major part of the office resources and involving constant overview and involvement from the senior partners and associates. Drawings were prepared by hand, along with fully detailed bending schedules for the reinforcing bars.

39. For a shear wall protected gravity load system building, the structural design process typically proceeded along two parallel paths: the design of earthquake resisting elements and the design of gravity load carrying elements. These could generally be designed independently of each other because, as I have already said, the gravity load carrying elements, such as floor beams and columns, did not have enough stiffness to attract significant earthquake loadings and therefore could be omitted from the earthquake analysis. Stiffness is a structural term that measures the amount of force

that it takes to deflect a structural element. So a big wall has much greater stiffness than a thin column.

40. My experience of working with Andy Buchanan on the Westpac Centre job, and the experience I gained, set me on the path to designing several multi-storey shear core buildings at Holmes Wood, although some of these did not proceed past the preliminary design stage. Two of the buildings I worked on had significant eccentricities due to the configuration of the shear walls.

41. The first of these buildings was a 14 storey building in Wellington next to the Plimmer Steps. I did the structural design work under the supervision of Russell Poole. At the time the building was called the AA Centre. This structure included an eccentric shear core to carry seismic loading, together with a perimeter frame that provided supplementary torsional resistance to counteract the eccentricity of the shear core. This was a complex structure due to the interaction of the shear core with the perimeter frame, which greatly complicated the design.

42. In my experience, the structural design and construction of multi-storey shear core buildings is complex. The reinforcing detailing is typically complicated in the lower storeys, which are designed to yield under earthquake loading. Special elements called coupling beams are often used to dissipate seismic energy. The diagonal reinforcing for these adds more complexity to the detailing and design. I mention this because the CTV building utilised a coupled shear wall on the south side. I will come back to this issue in the course of my evidence.

Alan Reay Consultants Ltd (ARCL): 1984-1985

43. In 1984 I joined ARCL. There were no other engineers in the office, apart from Alan Reay. Tilt slab buildings were Alan Reay's speciality and in this field he was clearly a very prominent designer. The systems he had developed for tilt slab buildings were very efficient with regard to use of materials and ease of construction.

44. In the mid 1980s, about the time I went to ARCL, the Christchurch CBD was beginning to see quite an increase in multi-level development. These buildings were usually reinforced concrete structures utilising shear walls. They required both experience and a relatively high level of expertise in design, compared with the more usual low rise tilt slab buildings in which Alan Reay had specialised.

45. Alan Reay told me at the time of my job interview that he had a couple of multi-storey jobs in the pipeline and I understood from discussions with him that my expertise was needed for these jobs. As far as I was aware the only multi-storey buildings ARCL had done prior to my arrival were Ibis House at 183 Hereford Street and possibly the Kamahi Towers apartment building in Carlton Mill Road. I think these were both 1970's concrete block structures. However concrete block was no longer an option for multi storey construction by the 1980s because it could not be made ductile in a practicable way for earthquake loadings.

Landsborough House

a. The design work

46. After I commenced work at ARCL Alan Reay confirmed that he had the job for what is now known as Landsborough House and also had the job for what is now called the Age Concern building. This is on the corner of Cashel Street and Cambridge Terrace. Landsborough House is an eight storey building on the north-west corner of Gloucester and Durham Street **[BUI.MAD249.0269.1].**

47. When I was shown the preliminary layout for Landsborough House I was surprised at the configuration of shear walls. There was a single wall along the north side, adjoined by several short interior walls at right angles alongside the services area. The configuration was eccentric to the north side of the plan to the extent that it was immediately apparent to me that it would not be a workable solution. I do not know how

this configuration was arrived at because I had no involvement with the design up to that point.

48. My immediate reaction was to suggest an alternative structure utilising a closed shear core as close to the centre of the building as possible. As was typical with all eccentric core buildings the reason for the offset configuration is to maximise the amount of lettable open floor area by moving the services and associated structure to the side, instead of the optimum position in the middle of the building. I would have preferred to have the shear core in the centre of the building, but it was clear to me that Alan Reay was committed to an offset configuration. As a compromise solution, after I had done my preliminary calculations I proposed that the shear core be offset on the middle of the north side, but still within the walls of the building.

b. The ETABS analysis

49. A final decision on this configuration was subject to testing by computer analysis using the ETABS programme, because the eccentricity was large. It was a code requirement to carry out this type of analysis for eccentric buildings more than 4 storeys high **[ENG.STA.0018.53]**. Throughout my evidence, unless I need to refer to one or other of the two relevant New Zealand Standards specifically, I will use the term "code" to refer to both NZS 4203:1984, which is the Code of Practice for General Structural Design, and Design Loadings for Buildings and NZS 3101:1982, which is the Code of Practice for the Design of Concrete Structures.

50. This requirement of an ETABS analysis was because with eccentric buildings it was the only means of accurately determining the likely response of the building to earthquake loading. It was also best practice at the time for determining the building deflections and design forces, which could not be accurately assessed by hand methods.

51. The task of carrying out the ETABS analysis was delegated to me by Alan Reay and I carried it out in the same manner as I had done at Holmes Wood, using ETABS on the

University of Canterbury computer. The first objective of the analysis was to determine if the structure had sufficient stiffness to limit the horizontal displacements or inter-storey drifts of the building to within the limits given in the code. This is the stage where the designer can gauge if the underlying design premise of a shear wall protected gravity load system remains valid. The deflection of the building is a function of the size, number and arrangement of the structural elements. If the analysis showed that the inter-storey drifts were too great then either the member sizes would be increased or the elements would be rearranged.

52. The results of the ETABS analysis of Landsborough House showed that the structural model worked, but the corner deflections were at or near the maximum code drift limits for the east-west, or eccentric, direction of loading.

53. At that time the ETABS analysis did not provide output results that could be used directly to interpret the deflections of the building. This was an important limitation on its use. The programme was basic, without the modern graphics features that provide ability to readily interpret the deflection output. Using the ETABS results to determine the deflections of a building required both experience in the use of ETABS and an understanding of the design of multi-storey shear core buildings. For example, at that time the ETABS deflections of the building were given in a single location at the centre of mass for each storey. The "raw" deflection data was given for the two horizontal x and y directions, or north and east in the case of CTV and Landsborough, which are both aligned the same way. The rotation of the building about its vertical axis was also given at the centre of mass, which is usually near the middle of a building of uniform plan. However, the maximum earthquake deflections are normally at the corners of the building where twisting due to any eccentric effects is at a maximum.

54. At that time calculation of the corner deflections required additional analysis to determine the centre of rotation of the building, which is not at the centre of mass when

the structure is eccentric. It is only after the centre of rotation has been determined that the corner deflections can be calculated.

55. It was essential to calculate the inter-storey drifts and be satisfied with the proposed structural configuration before proceeding with detailed design. This was the procedure that I learnt at Holmes Wood.

56. Before commencing detailed design, and as part of my review of the concept design for the shear core structure for Landsborough House, I also sought an overview comment from Professor Paulay at the University of Canterbury. I remained concerned about the proposed eccentric configuration of the shear walls and I wanted his opinion on the fundamental configuration with regard to the eccentricity and possible torsional effects. I was not looking for a detailed review.

57. I especially wanted Professor Paulay's opinion because I considered he was expert on torsional issues in building layout and reinforced concrete shear walls. I was aware from his lectures that in certain cases, depending on the torsional stiffness of the whole system, the response of some structural configurations can cause unexpectedly poor performance once ductile yielding has commenced under earthquake loading. Ductile yielding of structural elements is a key factor in limiting earthquake forces in buildings, but although the earthquake load is limited the deflection of the building is not. The energy dissipation occurs as a result of the building deflecting and at the same time, yielding the reinforcement. The more yielding and displacement, the more energy dissipation. The design codes utilise this aspect to dissipate seismic energy by controlled ductile yielding of the reinforcement in selected elements, such as the base of the shear wall and diagonal reinforcing beams in coupled shear walls.

c. The 1984 Loadings Code

58. The 1984 Design Loadings Code defined the ability of a structure to dissipate energy by yielding by the structural type factor "S". The Commentary to the Loadings Code,

clause C 3.4.2 (b) on page 42 of NZS 4203:1984, explains that the S factor takes into account the ability of the structural type to dissipate energy in a number of cycles, "....on the assumption that the bulk of the chosen energy dissipation members in all the principal resisting elements of a given structural type will participate in the dissipation of seismic energy" **[ENG.STA.0018.47]**.

59. The structural type may refer to individual elements if they vary throughout the building, or to the building as a whole if a uniform structural system is used. For uniform response and yielding of the building under earthquake loading a uniform structural type in the direction of loading under consideration would be ideal. However it is not always possible to have a uniform system. Combinations of structural types were common and the code made some provisions for that.

60. Clause C 3.4.2 (b) of the Commentary outlines a method for dealing with differing S values but warns that, as at 1984 this method was not fully researched and therefore should be used with prudence, particularly for buildings over 3 storeys high **[ENG.STA.0018.47-.48]**.

61. Professor Paulay had mentioned in his lectures the example of a building with a wall on each end and otherwise little torsional resistance, which could lead to the majority of the yielding occurring on one of the walls. I had a clear recollection of him saying this. The yielding demand is primarily made of the first wall to yield, rather than being shared between the two walls. This becomes more pronounced if there is unequal stiffness between the two walls. I will come back to this issue because it is my opinion that this example applies to the configuration of the north and south shear walls of the CTV building.

62. Professor Paulay did not raise any such fundamental issues with regard to Landsborough House, but he did comment on the eccentricity of the building and a possible loss in stiffness and consequent increase in deflections arising from cracking

of the shear walls under earthquake loading. I was aware of this possibility and I had used reduced properties in the ETABS analysis to take account of a loss of stiffness due to cracking in the relevant structural elements. This was my normal practice, but Professor Paulay still cautioned me about this issue.

d. Remaining concerns over the Landsborough House design

63. I discussed my concerns, and Professor Paulay's caution, with Alan Reay. He was dismissive of this aspect and we proceeded with the design. While I thought the design for Landsborough House was at the limit of acceptability, I believed that it met the code deflection requirements and as a result I accepted that the design would proceed.

64. My concern was whether the gravity load system would be adequately protected by the shear walls. I took heed of Professor Paulay's comments when it came to the detailing of the column hoop reinforcing in the end regions of the columns, I detailed the column tie reinforcing with a reasonable provision for some ductility demand in the end regions of all the columns, just in case deflections greater than those calculated in the ETABS analysis occurred in an extreme earthquake event. .

e. The indexed calculations

65. The formal calculations for the primary structure are usually bound in an indexed A4 set. However, other calculations are sometimes carried out on preliminary drawings or on the computer output and may not be bound together or filed with the indexed set. It was, and still is, normal to use judgement or experience, or traditional details, for some parts of the design. This was usually for less complex aspects or areas where the answer was known from another job.

66. In the course of preparing my evidence I have reviewed the calculations I did for Landsborough House, which I left with ARCL when I resigned. I have been advised by Counsel Assisting that the calculations I reviewed were obtained from ARCL. Most of

my important hand calculations have been included. This includes the preliminary checking of the corner deflections that I did for the design configuration I had been given by Alan Reay that showed me it did not work and led me to reconfigure the shear walls with a closed shear core **[BUI.DUR287.0003C.85-.91]**. The calculations I reviewed also include the further set of preliminary hand calculations I did that satisfied me that the corner deflections for my reconfigured walls were less than code maxima and that this was enough to proceed with the ETABS analysis **[BUI.DUR287.0003C.92-.100]**.

67. All of this work is included in the calculations I have reviewed for the purpose of preparing my evidence. However, the data I obtained from the ETABS analysis that I did for Landsborough House was not included with the information obtained from ARCL.

68. If David Harding had been following the calculations I left with ARCL the process he needed to go through to check the corner rotations resulting from the eccentricity of the CTV building should have been clear. However there is nothing in the CTV calculations that I have examined that indicates he did do this.

f. Alan Reay's involvement in the Landsborough design work

69. I have been asked by Counsel Assisting to describe Alan Reay's involvement in the design of Landsborough House.

70. I was very much in the driving seat in doing the structural design for Landsborough House. I had the sole responsibility for the ETABS work, including the analysis of the output. To assist me with producing the structural drawings, Terry Horn, a draughtsman from Holmes Wood who was experienced in detailing this type of building, was engaged by ARCL. I had no dealings with the client at all and few dealings with the architect in relation to project management over the Landsborough House job, but

carried out my role behind the scenes as the technical designer, specification writer and structural detailer.

71. I expect I would have shown Alan Reay the results of the analysis. He did make some comments on the design and gave some instructions to improve the method of construction by using some precast elements. The main instructions that I recall related to the use of precasting for the coupling beams to expedite the shear wall construction, which I agreed was a good idea, and the use of precast concrete fire separation walls between the egress stairs in the service core, which were then able to be lifted in for each floor. However, he was not closely involved in the work I was doing and had no involvement in the ETABS analysis.

g. Permit dates

72. I have been advised that the building permit application for the building is dated 6 June 1985 and the permit is dated 9 August 1985. I do not recall having any involvement with the building permit application process. My recollection is that this would have been about the time that I resigned from ARCL.

73. I cannot recall if construction of Landsborough House was underway at the time I resigned from ARCL. I had no involvement with the construction of the building.

Bradley Nuttall House

74. Alan Reay subsequently used the design of the Landsborough House building again for the Mair Astley building, now known as Bradley Nuttall House, situated in Cambridge Terrace. I will refer to this building as the Bradley Nuttall building. This was identical in plan to Landsborough House, but one storey lower. There was little needed in the way of structural design for this building, because the structural design already existed in Landsborough House. My involvement with the structural design was limited mainly to the design of the architectural precast spandrel panels that form the exterior façade of

the building. These were bolted onto the main structure as non-structural elements, that is, they are separated from the structure so as not to interfere with the main structure during earthquake deflections and they carry only their own weight.

75. I had no involvement with the client on the Bradley Nuttall job and little to do with recycling the Landsborough House structural design within the office. Alan Reay handled this with the draughtsmen. I cannot recall whether or not I left ARCL before documentation for this job was completed. I have been advised the building permit application date shown on the drawings is dated 18 July 1985 and the permit is dated 23 October 1985.

Age Concern Building

76. The other eccentric shear core building I was involved with at ARCL is now known as the Age Concern building on the corner of Cashel Street and Cambridge Terrace. As I have already mentioned, both this and Landsborough House were mentioned to me by Alan Reay as jobs that were in the pipeline when I was considering joining ARCL. I recall that this design was carried out concurrently with the Landsborough House design and utilised a similar configuration of walls for the shear core. I am advised that the building permit application is dated 2 April 1985.

77. I had no involvement in setting up the Age Concern job. I did do the detailed design. This building is a four storey reinforced concrete building. Alan Reay used his tilt slab expertise to design it with full height tilt slab walls forming the shear core. At the time this was unusual and extended the limits of this type of construction in Christchurch. Again, I believe the typical eccentric shear core was chosen for the purpose of optimising open lettable space.

78. There was no code requirement to used ETABS to analyse this building because this is only required under the code for buildings more than 4 storeys high. I was able to carry

out the calculations by manual methods. I do not recall ETABS being used for this job and I do not think it was. I carried out the calculations by manual methods.

Development of ARCL expertise with multi-level shear core buildings

79. I have been asked by Counsel Assisting whether the design work on the Age Concern building would have provided ARCL, and Alan Reay personally, with experience and expertise relevant to the design of a multi-level shear core building on the scale of the CTV building. In relation to the seismic analysis and shear core design it did not, for two reasons. First, I do not believe ETABS was used. Secondly, I did the detailed hand calculations for the shear core design and Alan Reay was not involved.

80. The Landsborough House, Bradley Nuttall and Age Concern buildings were all designed by ARCL on the basis of the knowledge about shear core buildings that I learnt at Holmes Wood and brought to ARCL during my time there. Although I believed the design of these buildings met the code at the time, they were all at the limit of what could be achieved with eccentric shear cores and there was no margin for error. My personal view was that these were not desirable structures to be designing. However I endeavoured to make the best of them given the constraints presented to me and to ensure that they complied with the code requirements.

81. I resigned after about one year of working at ARCL. During my time with Alan Reay I found that he preferred to work as the principal consultant with other design disciplines such as architects being engaged by him. He exercised tight control of the office and was very much in charge of the projects. I found that I was essentially relegated to the role of back room structural designer with my role limited to technical design and production of documents. This mode of operation did not suit me and I thought it was unlikely to change. I went to work with Lovell-Smith Cusiel on the design of what is now called the Holiday Inn building in High Street.

82. When I left ARCL there was no designer there who had experience of using either the ETABS system, or multi-storey shear core design.

The CTV Building design

83. I had no involvement with the design of the CTV building. When I left ARCL in early 1985 I had no knowledge of the building. As far as I knew, it was not even a possible job on the horizon.

84. In late 2011 I briefly discussed the CTV building with David Harding while chatting prior to a professional engineering meeting at the University of Canterbury. I know David Harding reasonably well from professional contact and I knew that he had worked for ARCL for a number of years before I went to ARCL. He then left and went to the Waimairi District Council. I also knew he had gone back to ARCL after I had left. He told me that he had done the calculations for the CTV building using the Landsborough House calculations as an example.

85. I knew about David Harding's structural engineering background and I was surprised to learn that he had been in the position of taking on the structural calculations for the CTV building. To the best of my knowledge David had no experience with the design of multi-level buildings prior to rejoining ARCL and I have been advised that David has confirmed this in correspondence with Counsel Assisting

86. I was concerned when David Harding told me he had followed my calculations for Landsborough House, for two reasons. Firstly, because it was unlikely that the calculations were sufficiently detailed for a "first time" designer to be able to adequately understand the design processes. The calculations did not record all my thinking processes or the decisions I had made on the basis of my judgement and experience. Secondly, although both the Landsborough House and CTV building designs were shear wall buildings that were eccentric for earthquake loading in the east-west direction, their shear wall designs were significantly different.

Differences between the CTV building and Landsborough House

87. I will now deal with these differences and the significance of them. Some of these were readily apparent to me from looking at the CTV building and the remnants of the structure after it collapsed. Others only became clear to me after a closer examination of the drawings and calculations.

(a) Differences between Landsborough House and the CTV building that are readily apparent

88. *Wall configuration*: The Landsborough House structure was designed to perform as a closed shear core or tubular structure. It was offset to one side of the building and therefore torsionally eccentric for loads in the east-west direction, but central and symmetrical for loads in the north south direction. The shear core provided the total shear and torsional earthquake resistance for the building. This is important for two reasons. First, a tube is more efficient at resisting torsion than an open walled configuration and a prime reason for using a closed shear core is to resist torsional forces arising from the eccentricity. Secondly, there were no other structural shear walls in Landsborough House that would influence the behaviour of the shear core under seismic loading. As a result its behaviour was reasonably predictable. This is a key aspect to appreciate when considering the wall system of the CTV building which had an inherent mismatch between the large stiff core located on the north side of the building and the smaller, more flexible coupled shear wall on the south side. The significance of this is discussed in more detail later in my evidence.

89. *Core location*: The Landsborough House shear core was eccentric but it was still within the main body of the building, whereas the CTV North core was on the outside of the building, which contributed to increased torsional effects. It also reduced the possible contact area for connection of shear walls with the main floor diaphragms.

(b) *Differences that are apparent from a closer examination of the drawings and calculations*

90. *North Core interaction with the South Shear Wall:* My review of the calculations for the CTV building shows me that they generally follow the path and the design process that I used in the calculations that I left behind from the Landsborough House work. However, the calculations I did for that building were specific to that type of shear core structure. They would not be fully applicable for anyone designing a different structure and, as I have already said, one of the most significant differences between the two structures related to the location of the north shear core and its interaction with the south shear wall.

91. To reiterate, the structure of the CTV building included a major arrangement of shear walls grouped to form an open sided core located at the north end, and a considerably less substantial coupled shear wall on the south end. For ease of description, I will refer to these as the North Core and the South Coupled Shear Wall. These were the two principal seismic elements in the east-west direction. However they were substantially mismatched in strength stiffness. This is significant because even with this mismatch, they each needed to carry about the same level of earthquake loading under the east-west earthquake. This is shown in the diagrams showing the distribution of ground floor shears on page S12 of the CTV calculations **[BUI.MAD249.0272.12]**.

92. The reason for this is that they were the only two shear wall elements acting in the east-west direction and they were approximately equally spaced about the centre of mass of the building. The effect is that half the load went to each end. However, due to the difference in their stiffness the South Coupled Shear Wall was going to deflect much more than the North Core and this would cause the building to twist about a vertical axis near the stiffer, north end of the building. This is shown on a separate diagram that I have prepared **[BUI.MAD249.0405]**.

93. This approximately 50:50 sharing of earthquake load between the two walls might appear to conflict with the expectation that a stiffer wall would attract a greater share of the earthquake load. However for it to do this the building had to be constrained against twisting. To achieve this there needed to be opposing walls in the orthogonal direction, i.e. the north-south direction. In the case of the CTV building there were four walls in the North Core which acted in the north-south direction, but these were not sufficient to achieve significant torsional restraint.

94. The ground floor shears shown on page S12 of the CTV calculations **[BUI.MAD249.0272.12]** demonstrate that the four north-south walls of the North Core have relatively small shear forces in them under the east-west earthquake loading. This is because they were relatively slender and open on one side. They were also close together compared with the size of the floor plan. The small forces shown in the calculations on page S12 show that these walls would not provide a substantial resistance to the torsional or twisting forces. The practical aspect of this was that the structural system being relied on for the east-west direction of the CTV building was primarily a system of two elements with unequal stiffness, the North Core and the South Coupled Shear Wall.

95. Without the South Coupled Shear Wall the CTV building would have had an eccentricity in excess of half the building. This would not have been workable. It appears that the South Coupled Shear Wall at the south end of the building was intended to counterbalance the eccentric effect of the North Core and this was the obvious place to locate such a wall. However, in my view that solution was still not enough because it would place the bulk of the ductility demand on the lesser of the two walls and the first wall to yield would continue to yield with increasing deflection and rotation of the building.

96. The imbalance between the North Shear Core and the South Coupled Shear Wall appears to have been increased during the design process by scaling factors applied to the design earthquake loading. The scaling has resulted in a significant reduction in the earthquake loading used by ARCL for the design of the South Coupled Shear Wall and this has significantly lowered the yield strength of that wall from the initial level using the Static assessment. While the South Wall was reasonably well detailed for ductility, and would have been able to sustain the earthquake loading by ductile yielding and plastic deformation, the onset of early yielding would have increased plastic deformation in the wall and increased lateral deflections of the south end of the building. The reasons for this are described in more detail later in my evidence.

97. The critical question for this design is whether or not the underlying design premise that the South Coupled Shear Wall shear was stiff enough to provide the necessary protection to the gravity load system remained valid for the chosen level of design load.

98. *Connection of Shear Core to Floor Diaphragms:* Because the CTV North Core was on the outside of the building the possible contact length with the main floor diaphragms was reduced to the approximately 4m where one area of floor slab extended into the core between the two western most wing walls. The two eastern most wing walls of the core had minimal connection where they contacted the floor diaphragms at their southern ends. Some of these connections were later upgraded in the upper levels as a result of a review by Holmes Consulting Group.

99. By contrast the Landsborough House shear core was within the body of the floor diaphragms. This had what I would call, a "spanner" effect that enabled the floor diaphragms to transfer torsional loads to the core, in addition to the connections provided by reinforcing from the walls into the floors along the east and west sides of the core and the connection where the two main north-south floor beams attached at the corners of the core.

100. *Location of Gravity Beams*: Landsborough House did not have gravity beams running in the critical east-west direction that would be vulnerable to deformation from inter-storey drifts during seismic loading and could induce significant unintended bending and shear forces. The absence of the east-west gravity beams meant that the columns were relatively free to rotate as intended, as pin ended struts, in the east west direction. This was consistent with the design premise of a shear wall protected gravity load system.

101. By contrast the CTV building had relatively stiff floor beams running in the east-west direction that were capable of providing unintended rotational restraint to the top and bottom of the columns and hence could induce unwanted seismic forces into the gravity load system if inter-storey deflections were greater than expected.

102. *Block boundary walls:* In Landsborough House there was a full height concrete block boundary wall abutting the core on the north boundary. This wall was specially detailed with a number of vertical joints to create slender wall elements to prevent them attracting seismic load that would significantly influence the shear core. These slender blockwalls were on the outside of the structure, not between columns and beams. In this location they could not become engaged with the gravity beams and columns. By contrast the CTV building had a block wall between floors on the west side extending up three storeys and located between the columns and beams, with a separation gap that may not have been sufficient to prevent the structure from engaging with the walls during earthquake loading. This may have affected the lateral response of the CTV structure, causing it to respond in an unintended way.

103. *Spandrel panels:* The Landsborough House spandrel panels were located well away from the gravity columns and separated so that they could not come into contact with the columns during earthquake deflections. Some of the CTV building spandrels were located close to the columns and could have come into contact with the columns during the earthquake.

104. *Column Reinforcing:* The Landsborough House column ties were 10mm diameter square hoops and they were more closely spaced in the column end regions to provide for some ductility under extreme earthquake loading. In my review of the Landsborough House drawings I have been able to confirm that typically the column ties shown there are 10mm diameter square hoops at 150mm centres in the end regions, adjacent to the floor and ceiling. These are the critical areas. In the less critical middle region the ties are 10mm diameter at 250mm centres. In the ground floor the columns have 10mm ties at 150mm centres over their full height.

105. This amount of column tie reinforcing did not make full provision for plastic hinging, but I considered it was reasonable at the time given that there were no floor beams to restrain the columns and induce bending moments into the columns in the critical direction.

106. By contrast the CTV building had R6 spiral ties at 250mm pitch, which is about 20% of the typical ties used in Landsborough House.

CTV Calculations

107. In the course of reviewing the calculations for the CTV building I have identified parts where ARCL has not followed the process that I used for Landsborough House. This is principally the calculations of the corner deflections. There are also some significant differences to the way I would have interpreted the code for determining the design earthquake loading and the application of the structural type factors, including the building period and the scaling factors used.

108. *Corner Deflections:* I have referred earlier in my evidence to the fact at the time the CTV building design was done the ETABS analysis did not provide output results that could be used directly to interpret the deflections of the building. It was essential to calculate the inter-storey drifts and be satisfied with the proposed structure

configuration before proceeding with detailed design. I had followed this procedure for Landsborough House.

109. The calculated east-west deflections shown in the ARCL calculations are smaller than I would have expected them to be. I saw nothing in the ARCL calculations that showed the rotation of the building had been taken into account to determine the maximum deflections at the south corners. There is no working of this in the calculations. Pages S15 and S16 of the calculations is where I would have expected to find this. **[BUI.MAD249.0272.15 and 16]**. It appears likely that the ARCL deflections are for the centre of mass and not for the maximum deflections at the south-east and south west corners of the building.

110. I initially did a preliminary check on corner deflections on the computer at work using the ARCL Microstran computer model of the South Coupled Shear Wall. This indicated significantly larger deflection at the South Wall than the deflections given on page S16 of the ARCL calculations **[BUI.MAD249.0272.16]**. As a result of that I sought more detailed information from Clark Hyland.

111. I contacted Clark Hyland and asked him if he could provide me the ETABS model corner deflections of the CTV building that he had calculated as part of the Consultants Report to DBH so that I could compare these with the deflections calculated by ARCL **[BUI.MAD249.0344A-F]**. After receiving this information I checked the corner and centre of mass deflections of the building. I have plotted the east-west deflections in graphic form for easier interpretation **[BUI.MAD249.0409]**. The graph shows that the south wall corner deflections calculated by Clark Hyland are substantially greater than those given in the ARCL calculations for the east-west direction. Significantly however, when the calculations done by Hyland and ARCL are compared on the basis of a centre of mass deflection they are much closer, although the east-west deflections are not as close as they are for the north-south **[BUI.MAD249.0412]**.

112. If this is correct the ARCL calculations would have underestimated the deflections. This may have misled ARCL in relation to the potential performance of the building for the chosen configuration of shear walls with the design being completed in the belief that the underlying design premise had been met, namely that the gravity load carrying system was protected against earthquake forces by the stiffness of the shear walls.

113. This would also explain why the gravity load beam-column frames did not have any special detailing for members subject to seismic loading, including the absence of column reinforcement for possible plastic hinging action.

114. *Design Earthquake Loading:* My concern about the design earthquake loading relates to the period of vibration of the CTV building and the scaling factors that were used. To explain this I need to comment briefly on how the loads are derived.

115. The design earthquake loading is derived from the NZS 4203:1984 Loading Standard. Under these provisions the simple method for determining the earthquake load for a regular building under four storeys high is called the Static load method. The Static load is a horizontal load applied over the height of the building with a bias towards the top of the building. It is calculated using a simple formula where the total earthquake load on the building is determined by a base shear coefficient Cd=CRSM. In this formula, C is the basic earthquake load determined as a function of the first period of vibration of the building. This is shown in the code as a graph and is called the response spectrum. The value of C is a proportion of gravity, or the weight of the building. For example a value of 0.1 means 0.1g or 10% of gravity.

116. The second factor, R, is the risk factor, which is 1.0 for most buildings of normal use.

117. The third factor, S, is the structural type factor. This determines the level of ductility that the building is to be designed for. Usually the S factor is set as 1.0 for the analysis and adjusted for each wall, depending on its height to length ratio, but within limits as

indicated in the Commentary to the code. Ideally the S factor would be kept the same throughout the structure, at least in each orthogonal direction.

118. For ductile or slender cantilever shear walls the S factor would be 1.0. For coupled shear walls, which are more ductile, the S factor could be reduced to 0.8, depending on the proportion of shear forces carried by the coupling beams in the wall. The South Coupled Shear Wall of the CTV building was such a wall, which was ductile enough for S=0.8. For large stiff walls the S factor would be increased to a maximum of 4, which reflected that it would behave in an elastic manner without significant ductile yielding. The North core of the CTV building was such a wall.

119. M is the material factor, which is 0.8 for reinforced concrete and constant for the whole building.

120. The period of vibration for the building is very important in establishing the basic earthquake load value of C. In the 1980's the rule of thumb method for quickly estimating the period of vibration for Christchurch earthquake loading was 0.1 seconds times the number of stories. For example, a six-storey building such as the CTV building had a first period of vibration of 0.6 seconds.

121. The shape of the response spectrum graph in the 1984 code, that applied to Christchurch was such that for any period of less than 0.7 seconds the earthquake load is constant on a plateau at a maximum level. Beyond 0.7 seconds the earthquake loading reduces linearly with period until the value of 1.2 seconds where the graph flattens off.

122. For the CTV building the initial period assumed in the ARCL calculations was less than 0.7 seconds and the structural type factor assumed was 1.0. With the importance factor as 1.0, and the material factor as 0.8, this resulted in a base shear coefficient of Cd = CRSM = 0.125 x 1.0 x 1.0 x 0.8 = 0.10. This resulted in a total base shear of 3300KN.

123. The ARCL calculations I have reviewed show that the period of vibration found by the ARCL ETABS analysis had increased the first period of vibration to 1.06 seconds and the earthquake loading was reduced accordingly by approximately 30%. This resulted in a base shear coefficient of 0.071 and a base shear of 2350KN.

124. This period of 1.06 seconds was consistent with the DBH report findings which found the building period to be 1.03 seconds in the east-west direction and 1.2 seconds in the north-south direction. However, it differs significantly from what I would have expected based on experience with other shear wall designs. These tended to have a period of vibration consistent with the rule of thumb of 0.1 seconds x the number of storeys.

125. When I did my Landsborough House design I did an initial manual calculation using this rule of thumb approach which gave a period of 0.7 seconds **[BUI.DUR287.0003C.83]**. I then did a further calculation based on NZS 4203:1984 C3.4.4.1 **[BUI.DUR287.0003C.91]**. My experience with Landsborough House and other stiff shear wall buildings I have worked on has confirmed for me that these buildings generally do perform in a manner consistent with that rule of thumb assessment. Applying that to the CTV building, this would have led me to stick with the initial assessment of 0.7 seconds.

126. I am also surprised that the longer period of 1.06 seconds was used for the CTV building because it is not consistent with the stiffness of the North Shear Core. The information supplied to me by Clark Hyland, to which I have previously referred **[BUI.MAD249.0344A]** shows that the first mode of vibration for the east-west direction was dominated by the response of the South Coupled Shear Wall, not the North Core which had a considerably shorter period of vibration of 0.32 seconds.

127. My concern is that the design load was reduced on the basis of a period of vibration appropriate to the response of the South Coupled Shear Wall and that the lower

loading derived from this longer period would reduce its design strength relative to the North Core, which was inherently overstrength due to its large size.

128. *Scaling factors:* The code provides for a Scaling Factor to be applied to the ETABS results so that they remain within limits controlled by the simple Static load case. This is because for the ETABS analysis the earthquake loading is determined in a different, more complex way, using a greater number of higher modes of vibration for the building.

129. This method of analysis is called an Elastic Response Spectrum Analysis, or ERSA for short. The normal approach is to use the first three modes in each of the principal directions, these being x, y and z. These are the axes corresponding to both the horizontal orthogonal directions and the vertical axis of the building. The complication with using a number of modes is how to combine the modes in such a way that the behaviour of the building is represented reasonably realistically. The differing modes may be acting in different directions at any given point in time, so they may cancel or they may add together depending on the nature of the earthquake.

130. The code specifies the method for combining the forces from the selected higher modes. In 1984 the code method used to combine the forces resulting from the higher modes was called the square root of the sum of the squares. (SQRSS). This eliminated any negative values from the equation and allowed them to be combined as a positive sum, and then to take the square root for the final answer.

131. It was often found using ETABS that this SQRSS method of combining the forces resulted in a lower level of overall load on the building model than would be determined using the simple Static Loading method. The code allowed the ERSA results to be lower than the Static results, but limited to 90% of Static load for the whole building and 80% of Static load for any one storey. The reason for allowing this is given in the Commentary to NZS 4203:1984 clause C3.5.2.4: "When a building is designed to resist

the more accurate distribution of loads given by the spectral modal analysis then an improved performance will result. Base shear values are therefore reduced to 90 percent of the values given in section 3.4" **[ENG.STA.0018.60]**. Section 3.4 is for derivation of the Static load case.

132. My practise in using the scaling factor was to use it to set a lower bound in order to ensure that the forces determined from the ERSA method were not too low relative to the Static method. In effect, the Static Load case was used as a baseline or control value against which the ERSA results could be calibrated.

133. *Scaling adjustment to the ERSA results:* The scaling factor K was determined by comparing the Static base shear, or total Static earthquake load, with the ERSA base shear. If the ERSA base shear was less than 90% of the Static base shear it was scaled up. Similarly, if at any particular storey level the ERSA shear was less than 80% Static, it was scaled up. If the ERSA base shear was greater than 90% Static then the results could be scaled down, but with caution. In my experience there is a closer correlation between the Static and ERSA results with a symmetrical building than there is with an eccentric building where there can be quite significant differences.

134. On my review of the Landsborough House calculations I can see that I could have scaled the ERSA results down on the basis of base shears, but I chose not to do this and used the higher level of forces for the Static base shear. This produced design forces for the critical walls that were higher than any of those given by the ERSA analysis. By contrast, the ARCL calculations show that for the CTV building the ERSA results were scaled down **[BUI.MAD249.0272.17]**.

135. For the critical east-west direction, or the y earthquake as it is called in the ARCL calculations, the scaling factor K was determined as 0.76 based on the longer period of 1.06 seconds. I would not have done this because of the mismatch in stiffness between the North Core and the South Coupled Shear Wall.

136. The ARCL calculations record on page S17 that a further SM=0.8 factor was then applied to the 0.76 that had been arrived at by the scaling factor K (S=1.0 and M=0.8 at that stage). It appears that this was done for the purpose of bringing the ERSA results into line with the Static Load case because the Static included the material factor 0.8, whereas the ERSA analysis did not. I would need to see the ETABS input data to confirm this. If this was done this would have led to a double application of the material factor M=0.8, which was implicitly already included in the initial scaling to 1.07.

137. The effect of these interpretations for the scaling factor resulted in scaled ERSA forces that were less than 0.8 Static, as shown in the summary table on page S18 of the calculations **[BUI.MAD249.0272.18]**. From that summary it appears that the greater design forces for 0.8 Static were chosen for the design of the shear walls because they were greater than the scaled down ERSA values.

138. The result of the ARCL scaling process is that the higher ERSA results were not used and the design loads for the shear walls were set at 0.8 Static load case. This is less than the minimum load level set by clause 3.5.2.4.1 of NZS 4203:1984 in terms of the base shear of the building so that the minimum global load of the whole wall is not less than 0.9 Static when using ERSA loads **[ENG.STA.0018.60]**.

139. *Structural type factor:* For the final design of the South Coupled Shear Wall a structural type factor of S=0.8 was applied to the 0.8 static results, resulting in a load level of 0.64 of the Static load. In summary and in numerical terms, this reduced the original 2000KN Static base shear on the South Coupled Shear Wall, derived from the original analysis using period =0.7 seconds, by 0.712 on the basis of the longer period of 1.03 seconds, to 1424KN. This was further reduced by SM=0.8x0.8=0.64 to 912KN and the factoring is shown in page S29 of the ARCL calculations as 0.57Staticx0.8=0.456, i.e. 46% of the original Static load for period of 0.7 seconds. **[BUI.MAD249.0272.29]**

140. The practical significance of these rather detailed calculations I have referred to is that this reduction of load leads to a corresponding reduction in the reinforcing requirements for the South Coupled Shear Wall. This in turn reduced the stiffness of the South Coupled Shear Wall because the stiffness is a function of the amount of reinforcing. In other words, the South Coupled Shear Wall was softer than it would have been if the higher earthquake loading had been used. This increased the susceptibility of the south end of the building to increased lateral deflections under east-west earthquake loading.

141. This increased the imbalance in the building because it effectively only applied to the South Coupled Shear Wall and not the much stiffer and stronger North Core.

142. Given that the load demand on each of these elements was shown in the ARCL analysis to be similar under east-west loading, the earthquake load on the whole building would have been largely governed by the yielding of the South Coupled Shear Wall. Once it yielded the system would essentially be limited to that load level. Any higher level of load would cause the building to rotate about the North Core.

143. *Signals of irregularities in the output data:* There were some strong signals in the ETABS analysis output data indicating irregularities in the structural model that should have alerted an experienced designer and triggered questions and further investigation into the behaviour of the structural model.

144. These included the longer building period that I have referred to earlier in my evidence of 1.06 seconds produced by ETABS, the ERSA base shear being larger than the Static base shear, and the contrasting difference in the S factors between the North Core wall and the South Coupled Shear Wall, given their approximately 50:50 load demand.

145. In addition there were some strong signals in the ETABS deflection data indicating that something was irregular with the model or the structural concept. These are first, the

deflections given in the ARCL calculations show that the building would have deflected 4 to 5 times more in the north-south direction than the east-west direction under their respective earthquake loadings and, secondly the ARCL deflections were at the code limit for the north-south direction, but not the east-west direction and were relatively small for the east-west direction. For me this would have indicated a possible error or inaccuracy in modelling the walls in the ETABS model and the need for closer consideration to check out the disparity between the two directions.

Conclusion

146. I can see from the calculations that the CTV building was designed on the basis of the underlying premise that the gravity load elements of the building would be protected against excessive lateral deflections and earthquake forces by the stiffness of the primary seismic shear walls. If the intended deflection limits implied by this design premise were not met, then the gravity load system of the building could be vulnerable to damage and instability in the event that earthquake deflections exceeded those anticipated.

147. In this respect I believe that the eccentric and unbalanced structural configuration of the CTV building, and the characteristics that I have described in my evidence, made it susceptible to increased lateral deflections under severe earthquake loading in the east-west direction.

Christchurch City Council: 1992-1995

148. In 1992 I decided to take a salaried position with the Christchurch City Council where I worked in the Building Control Unit from 1992 to 1995. The Council was looking for a structural engineer to assist in the transition from Local Government Bylaws to the Building Act 1991, which resulted in a major reorganisation of the Council Building Inspectors, Service Centres and Building Consent processes. I worked alongside Bryan Bluck and Graeme Tapper during that process. Bryan Bluck was the Head of the

Building Control unit at this time. Graeme Tapper was the senior building engineer and he reported to Bryan Bluck.

149. Counsel Assisting has asked me to describe the Building Control processes used by Graeme Tapper and Bryan Bluck and their interaction with consulting engineers in the course of building consent applications, in particular interaction with Alan Reay and ARCL during my time at the Council.

150. The role of a reviewer in the Council Building Control unit required ongoing interaction and liaison with the structural engineering community in relation to the building consent processes. I worked closely with Graeme Tapper in reviewing the structural engineering aspect of building consent applications. He taught me the bureaucratic processes and I assisted him with detailed technical matters as I was more up to date with the engineering Codes and design methods than he was.

151. I became aware after I joined the Council that Alan Reay, and ARCL building consent applications were causing the Building Control staff a lot of concern because of particular structural details used in the designs. It was not uncommon for ARCL jobs to be closely queried by Graeme Tapper and held up because he was not satisfied with the responses that he got from ARCL about these details. I found that Alan Reay, and ARCL, did not like Graeme Tapper's close scrutiny of their work. It was not uncommon for Alan Reay to go directly to Bryan Bluck to obtain the release of a building consent when he could not get approval from Graeme Tapper.

152. A number of technical disputes arose in relation to ARCL building consent applications during my time at the Council and I observed first hand the manner in which these disputes were handled within the Building Control Unit. On a number of occasions, they led to disagreements between Graeme Tapper and Bryan Bluck, with Graeme Tapper ultimately being overruled by Bryan Bluck on ARCL permits.

153. Bryan had a wider, more diplomatic role including reporting to the Council and a long history of reasonably good public relations with the consulting engineering community and because of that role he always made himself available to consultants.

154. In my experience working with Graeme Tapper I thought that he usually had the correct technical grounds for raising his concerns. He was a competent senior engineer. He had developed a good sense of the potential weak points in a structure. He had done civil engineering quality assurance work on the Benmore Dam and had worked as a structural engineer for Royds Garden, a well respected Southland firm. I believe he had good experience and training. This was evident in the way that he carried out his work.

155. However he could be confrontational when dealing with the consultants. He maintained high professional and ethical standards and had little tolerance for consulting engineers who submitted poor details or incomplete work. This would often result in difficult situations, which Bryan Bluck then had to deal with.

156. My observation was that part of the reason for Bryan Bluck overruling Graeme Tapper at times was that Bryan did not have a sufficient understanding of the technical matters involved to be able to confidently support Graeme Tapper. Based on my review of the 27 August 1986 letter that Graeme Tapper wrote to Alan Reay Consulting Engineer expressing concern about aspects of the structural design, I can see that there were particularly detailed technical matters involved and I do not think Bryan Bluck would have known enough of the technical details of the code to determine whether the aspects queried by Graeme Tapper met the code or not.

157. Bryan Bluck's attitude was that the consulting structural engineers were the experts and therefore the responsibility for code compliance lay with them, not the Council. On occasions, and under pressure, I observed that he tended to let the consulting engineers have the last say.

158. In the course of preparing my evidence I have been shown a handwritten letter from Graeme Tapper to Alan M Reay Consulting Engineer, which is a Council request for further information in relation to the building permit application for the CTV building. It is dated 27 August 1986: [**BUI.MAD249.0141.13]**. In my experience it was not unusual for Graeme Tapper to communicate in writing. When he was concerned that he would be overruled he would often say that he wanted to leave a paper trail.

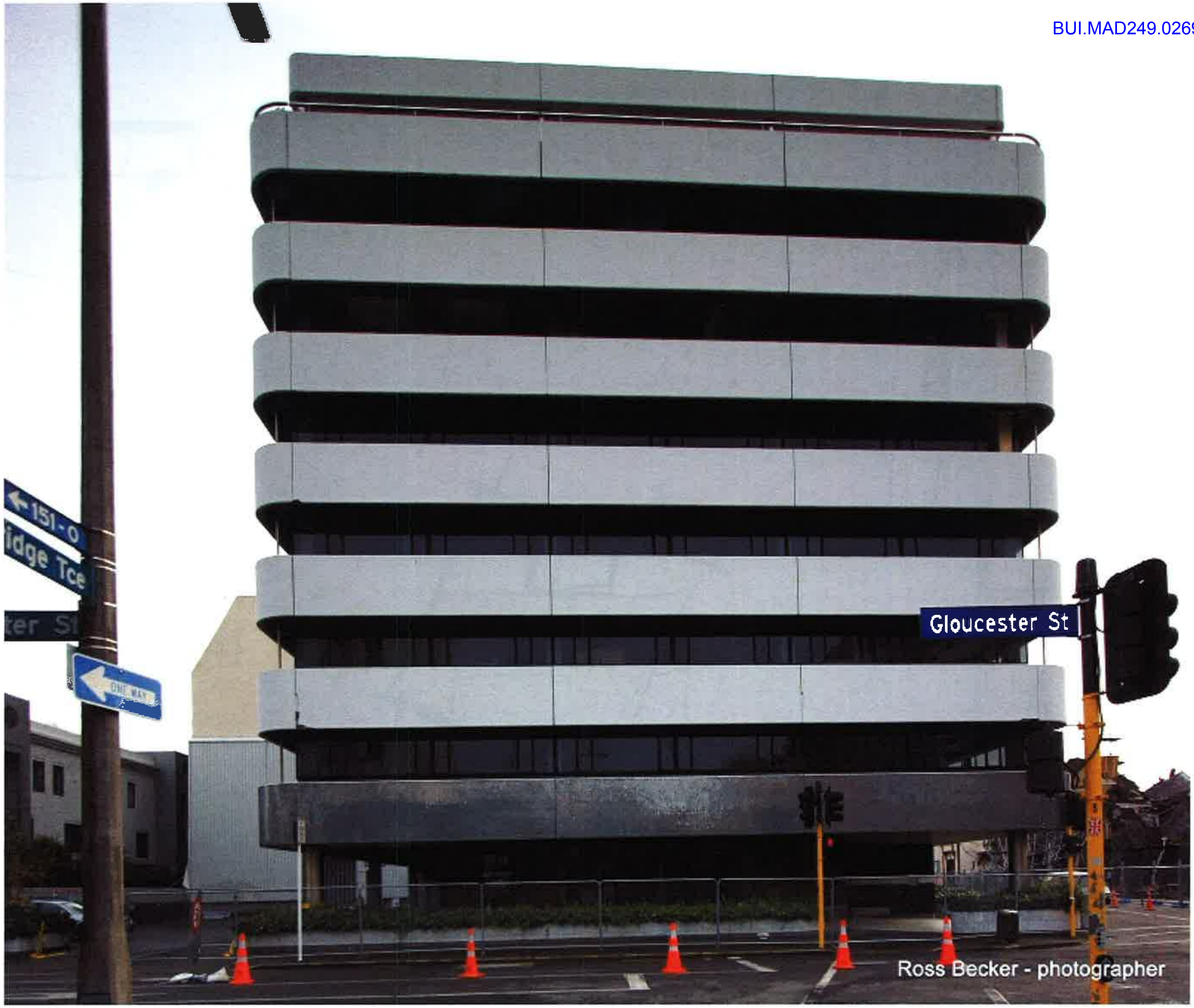
159. Graeme Tapper's letter identifies a number of concerns with the documentation provided to the Council and also with some of the structural detailing. This includes a reference to drawings S15 and S16, which show the floor connections to the shear wall system. Graeme Tapper has identified a concern about the mesh not providing adequate restraint to the steel tray deck Hi Bond flooring system for fire rating purposes. He also refers to "*...general connections between floor slab and walls..and the stirrups for the columns".*

160. I have examined the structural drawings for the CTV building. A number of the issues raised by Graeme Tapper would have caused me concern as well. This includes the shear wall connections.

Signed: John Henry

John Malcolm Terrence Henry

Dated: 22 May 2012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Ross Becker - photographer

force resisting system is equal to or greater than 3, then 0.1 V shall be considered as concentrated at the top storey and the remaining 0.9 V shall be distributed in accordance with equation 30.

(b) For chimneys and smoke-stacks resting on the ground, 0.2 V shall be considered as concentrated at the top and the remaining 0.8 V shall be distributed in accordance with equation 30.

(c) For buildings with set-backs the load distribution shall comply with clause 3.4.11.

C3.4.6.1 (d) The provisions of clause 3.4.6.1 (d) will result in base shears similar to those that would be given by the equivalent static force method of analysis, but the distribution of forces will be more appropriate to the particular features of the irregular structures.

Nevertheless it will be necessary to make an equivalent static force analysis for irregular structures in order to obtain the limiting values in clauses 3.5.2.4 and 3.5.2.5. The term "major buildings" is intended to exclude low buildings that do not warrant the more complex procedure of a dynamic analysis. See also section 3.1.

(d) The distribution of horizontal seismic forces in major buildings that have highly irregular shapes, large differences in lateral resistance or stiffness between storeys, or other unusual structural features shall be determined in accordance with the dynamic analysis procedure of section 3.5.

3.4.6.2 At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution at that level.

3.4.6.3 Floors and roofs acting as diaphragms and other principal members distributing seismic forces shall be designed in accordance with clause 3.4.9. Allowance shall be made for any additional forces in such members that may result from redistribution of storey shears.

*3.4.7 Horizontal torsional moments

C3.4.7.1 Horizontal torsional effects are difficult to estimate. Both excitation and response are known with far less certainty than for translational behaviour. The effects are important however; a number of failures have been caused by horizontal torsion particularly at the ends and corners of buildings, and at re-entrant angles.

A designer's first aim should be to achieve symmetrical structures of similar resisting elements.

Three types of design approach are considered in this standard: a wholly static approach; a combined approach in which the vertical distribution of horizontal forces is given by a two-dimensional modal analysis (clause 3.5.2.2.1) and torsional effects are obtained from the static provisions of clause 3.4.7, and a three-dimensional spectral modal analysis (clause 3.5.2.2.2).

The static method given in clause 3.4.7.2 is intended to apply to reasonably regular buildings such as square, circular, or rectangular structures which have no major re-entrant angles and which are substantially uniform in plan.

Structures of moderate eccentricity are those for which the torsional component of shear load in the element most unfavourably affected does not exceed three quarters of the lateral translational component of shear load.

3.4.7.1 The applicable method of design for torsional moments shall be:

(a) For structures not more than four storeys high or for reasonably regular structures more than four storeys high which are symmetric or of moderate eccentricity, horizontal torsion effects shall be taken into account either by the static method of clause 3.4.7.2, or by the two-dimensional modal analysis method of clause 3.5.2.2.1 (which also uses clause 3.4.7.2), or by the three-dimensional modal analysis method of clause 3.5.2.2.2.

(b) For reasonably regular structures more than four storeys high with a high degree of eccentricity, horizontal torsional effects shall be taken into account either by the static method of clause 3.4.7.2, or by the two-dimensional modal analysis method of clause 3.5.2.2.2. However, it is recommended that the three-dimensional modal analysis of clause 3.5.2.2.2 be used for such structures.

(c) For irregular structures more than four storeys high, horizontal torsional effects shall be taken into account by the three-dimensional modal analysis method of clause 3.5.2.2.2.

Category 2b: *Storage and distribution facilities for products such as LPG, CNG, Natural Gas and other highly flammable, explosive, or poisonous materials in urban areas.*

Category 3a: *Central and local government facilities of particular importance following disaster;*
Defence establishments;
Hospital and medical facilities that are not essential facilities;
Electricity and gas supply authority facilities;
Prisons and other places of restraint;
Post Offices (major);
Airport buildings.

Category 3b: *Major art galleries, museums, libraries, and archival record depositories.*
It is recommended that buildings of particular cultural significance be designed to this Standard.

Category 4: *All other buildings.*

(b) Structural type factor S *(see tables 5 and 5B)*

The structural type factor S is intended to reflect the potential seismic performances of different structural systems. The specified level of S primarily takes into account the ability of the structural type concerned to dissipate energy in a number of load cycles, and secondarily its degree of redundancy where appropriate, on the assumption that the bulk of the chosen energy dissipating members in all the principal resisting elements of a given structural type will participate in the dissipation of seismic energy.

Where the earthquake resistance of a building must be provided by a combination of structural types, the designer is required to select an appropriate value for S by rational deduction from table 5, and this will necessitate consideration of the degree to which the various elements will contribute to the dissipation of seismic energy in severe earthquakes.

A method of determining rational design actions for buildings having horizontal force resisting systems in parallel, with differing S and M values in the direction being considered, is as follows:

Analyse the building, including torsion allowances, assuming S and M equal 1 for all sub-assemblies and then design each using the load effect derived from this analysis and modified by multiplying it by the S and M values appropriate to the sub-assembly.

This method is based on the premise that the values of S and M for a given sub-assembly should reflect its available ductility. As at 1984, this method has not been fully researched and therefore should be used

Table 5 STRUCTURAL TYPE FACTOR *S*

Item	*Description*	*S*
1	Ductile frames	0.8
2	Ductile coupled shear walls:	
	(a) $A > 0.67$	$0.8Z \leq 1.6$
	(b) $A < 0.33$	$1.0Z \leq 2.0$
	(c) $0.33 \leq A \leq 0.67$	By linear interpolation between 2(a) and 2(b)
3	Ductile cantilever shear walls. Single-storey ductile columns	
	(a) Two or more elements linked together	$1.0Z \leq 2.0$
	(b) Single element	$1.2Z \leq 2.0$
4	Frames of limited ductility of maximum height four storeys or 18 m or, with top storey, roof and wall mass less than 150 kg/m², five-storeys or 22.5 m. Cantilevered shear walls of limited ductility.	2.0
5	Buildings with diagonal bracing:	
	(i) Capable of plastic deformation in tension only:	
	(a) Single storey	2.0
	(b) Two or more storeys	2.5 or by special study
	(c) More than three storeys	By special study
	(ii) Capable of plastic deformation in both tension and compression.	1.6 or by special study
6	Single-storey cantilevered buildings supported by face loaded walls constructed of reinforced masonry or concrete.	2.0
7	Elastically responding structures:	
	(a) Reinforced concrete	5.0
	(b) Reinforced masonry	4.0
	(c) Prestressed concrete	5.0
	(d) Steel	6.0

Where A is the proportion of total overturning moment resisted by all beams (moments referred to the centroidal axes of all walls) and where:

Z = $3.0-h_w/l_w$ subject to $1 \leq Z \leq 2$

h_w is the height from base of the wall to top of uppermost principal storey.

l_w is the horizontal length of the wall in the direction of the applied load.

with prudence particularly for buildings over 3 storeys high. It is noted that this method will generally result in a modified base shear V as prescribed by equation 27.

Other points to note about tables 5 and 5B are:

Chimneys can come under one of the items 1 to 8 but very tall, slender structures, where second and higher mode effects can become significant, come outside the code and must be designed by special study. The same applies to tanks, but depending on the size and proportion of the tank, the sloshing action of the content will become significant in loading in which case they must be designed by special study.

(i) Item 1: *The structural type factor of 0.8 applies to all ductile frames subject to capacity design procedures.*

(ii) Item 2: *Requirements for coupling beams are given in clause 3.3.4.1, and requirements for shear walls designed for ductile flexural yielding are given in clause 3.3.4.2. As the proportion of total base overturning moment resisted by the beam diminishes, the structural type factor increases, in recognition of the degree to which energy dissipation in the more vulnerable elements, the walls, is concentrated.*

For the situation where only two walls are present,

$A = T\ell/M_o$

Where

T *is the axial load induced in the walls by the coupling beams,*

ℓ *is the horizontal length between the centroids of the walls,*

M_o *is the total overturning moment at the base of the structure due to the same loads used in the determining of* T.

(iii) Item 3: *Single storey ductile columns provide seismic resistance by cantilever action.*

(See also item 2 of table 9.) Requirements for shear walls designed for ductile flexural yielding are given in clause 3.3.4.2. The parameter Z, *incorporating the wall aspect ratio* hw/lw *is introduced in recognition of the reduced energy dissipation, for a given displacement occurring where shear effects are significant such as in squat walls.*

(iv) Item 4: *Requirements are given in clause 3.3.6.*

(v) Item 5(i): *Design requirements are given in clause 3.3.5. The deformation modification factors given in clause 3.8.1 take account of the characteristics of these structures.*

(vi) Item 5(ii): *These systems, when suitably designed and detailed, may give reduced displacement respon-*

Table 5B

***SM* or $S_p M_p$ FACTORS FOR TIMBER**

Item	*Description*	*SM or S_pM_p*
B1	Shear walls or diaphragms:	
	(a) Ductile	1.0
	(b) Ductile and stiffened with elastomeric adhesive	1.0
	(c) Limited ductility fixed with elastomeric adhesive	1.2
B2	Moment resisting frames:	
	(a) Ductile with an adequate number of possible plastic beam hinges	1.2
	(b) As for item B2 (a) but with connections of limited ductility	1.5
B3	Diagonally braced with timber members capable of acting as struts or ties:	
	(a) With ductile end connections	1.7
	(b) With end connections having limited ductility	2.0
B4	Elastically responding structures	2.4

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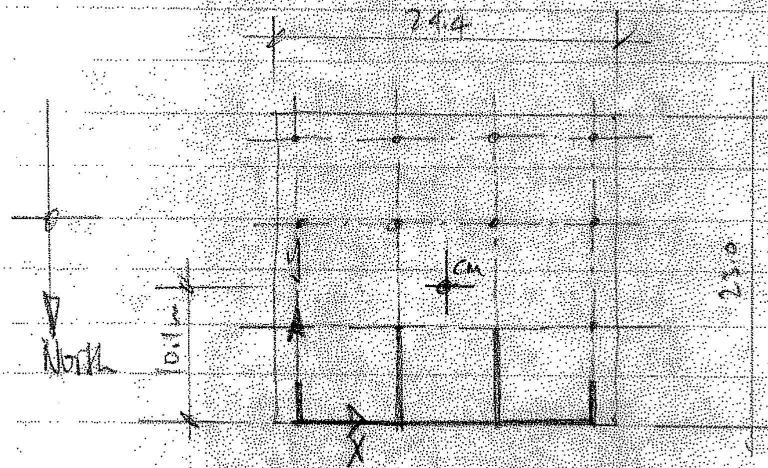
Torsional Eccentricities



Centre of Mass of Typical floor

All masses symmetrically located N-S axis

About E-W axis.

Item	Mass	y (from X axis)	My
Floor Slabs	2089	11.5	24023
Spandrel	146	23	3358
	276	11.5	3174
Beams.	326	14.4	4694
Rear wall	375	0	0
Shear walls	184	1.5	276
	332	3.6	1195
Columns	162	15.8	2560
Curtain wall	37	21.6	799
	60	11.5	690
Core	138	6	828
	4125		Σ 41597

$\Rightarrow \bar{y} = \frac{41597}{4125} = 10.1$ m

Centre of Rigidity

In Y direction - Symmetrical
$\Rightarrow$ CM & CR concurrent
$\Rightarrow$ $e_s = 0$

In x direction - assume shear centre is at North bldg wall
$\Rightarrow$ $e_s = 10.1 - 0.1$
$= 10.0$ m

Design eccentricity $e_d = e_s \pm 0.1b$

$\therefore$ e_{dx} for EQ in Y direction

$e_{dx} = \pm 0.1 \times 24.4$
$= \pm 2.44$ m.

and e_{dy} for EQ in X direction
Max^n $e_{dy} = 10.0$ m $+ 2.3$
$= 12.3$ m
Min^m $e_{dy_2} = 10.0$ m $- 2.3$
$= 7.7$ m.

Thus Torsional moments

T_y due to $e_{dx} = V_E\, e_{dx} = 3074 \times 2.44$
$= 7500$ kN-m

T_x max " " $e_{dy\,max} = 3074 \times 12.3 = 37810$ kN-m

T_x min " " $e_{dy\,min} = 3074 \times 7.7 = 23670$ kN-m.

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Relative Stiffness of Shear Walls

Y Direction - 4 walls
- 2 @ 3.0m x Say 400 avge — A & D
- 2 @ 7.2m x 300 avge — B & C

$$k = \frac{1}{\frac{L^3}{3EI} + \frac{3L}{EA}}$$

For all walls take $L = 26.4$ m.

Walls A & D: $A = 3 \times .4 = 1.2\ m^2$
$I = \frac{.4 \times 3^3}{12} = 0.9\ m^4$

B & C: $A = 7.2 \times .3 = 2.16\ m^2$
$I = \frac{.3 \times 7.2^3}{12} = 9.33\ m^4$

$$K_{A,D} = \left(\frac{26.4^3}{3 \times 2.5E7 \times 0.9} + \frac{3 \times 26.4}{2.5E7 \times 1.2}\right)^{-1}$$

$= 3633$ kN/m — $\frac{k}{\Sigma k} = .1092$

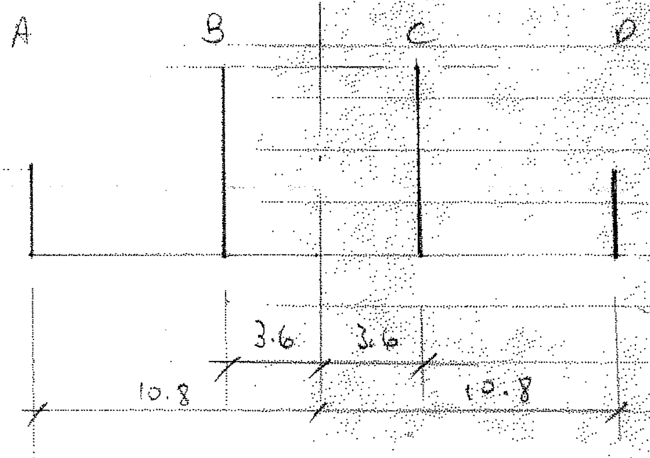
$$K_{B,C} = \left(\frac{26.4^3}{3 \times 2.5E7 \times 9.33} + \frac{3 \times 26.4}{2.5E7 \times 2.16}\right)^{-1}$$

$= 36021$ kN/m — $\frac{k}{\Sigma k} = .908$

Ratio $\frac{K_{B,C}}{K_{A,D}} = \frac{36021}{3633} = 9.9$

CALCULATIONS

Torsional Shears



$$V_T = \frac{kr}{\Sigma kr^2} \times T$$

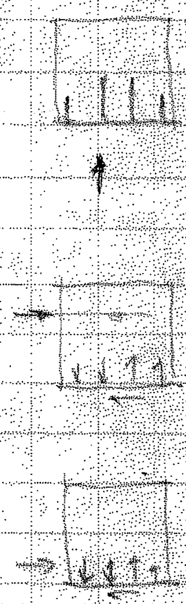
Element	k	r	kr	kr^2	$\frac{kr}{\Sigma kr^2}$	$\times V_T$ (37810 kN)
A	3.63	10.8	39.2	423.4	.022	831 kN
B	36.0	3.6	129.6	466.6	.073	2760 kN
C	36.0	3.6	129.6	466.6	.073	2760 kN
D	3.63	10.8	39.2	423.4	.022	831 kN
				1778		

E.Q	Torque	Shear A,B	Shear C,D	
V_y	7500	165	548	kN
V_x max	37810	831	2760	kN
V_x min	23670	520	1728	kN

CALCULATIONS

Combined Direct + Torsional Shear

Y EQ	A	B	C	D
Direct	141	1396	1396	141
Torsional	165	548	− 548	− 165
	306	1944	848	− 24



V_{12}

	A	B	C	D
X EQ emax	+ 831	− 2760	+ 2760	+ 831
X EQ emin	− 520	− 1728	+ 1728	− 520

Yielding of shear walls A and D governed by X EQ with max^n eccentricity.

Consider approximately overstrength shears

take $\phi_o = 1.5$, $\omega = 1.5$ } 2.25

A, D $\quad V_o = V_E\,\omega\,\phi_o = 831 \times 2.25 = 1870$ kN

C, B $\quad V_o = 2760 \times 2.25 = 6210$ kN

$v_i \leq (0.3\,\phi_o\,S + 0.16)\sqrt{f'_c}$ $\quad$ take $f'_c = 35$ MPa

$= (0.3 \times 1.5 \times 1.0 + 0.16)\sqrt{35}$

$= 3.6$ MPa

$v_i\ A, D = 1870 \times 10^3 / 0.8 \times 3000 \times 400 = 1.95$ MPa

$v_i\ B, C = 6210 \times 10^3 / 0.8 \times 7200 \times 300 = 3.59$ MPa

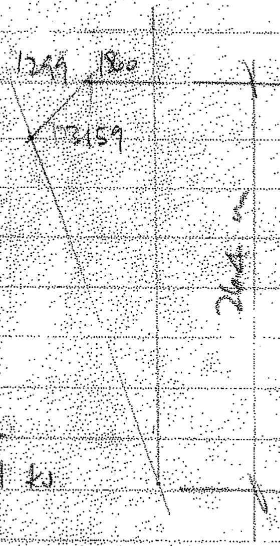
OK

ALAN M. REAY Consulting Engineer

Check Period.

$T = 0.063 \sqrt{\Delta}$ Δ = defl^n in m under load W_n

Level	W_x	h_x/h_n	W_n
R	1860	1.0	1860
7	4125	.875	3610
6	|	.75	3094
5	|	.625	2578
4	|	.5	2062
3	|	.375	1547
2	|	.25	1031
1	4125	.125	515
			16297 kN



Δ for straight cantilever

$$= \frac{11 W a^3}{60 EI} - \frac{PL^3}{3EI}$$

$W = 16297 + 1299$ $P = -1299$
$= 17596$ kN

In Y direction $\Sigma I = 2 \times (9.33 + .9) = 20.5 \text{ m}^4$

$$\Rightarrow \Delta = \frac{26.4^3}{2.5E7 \times 20.5}\left(\frac{11 \times 17596}{60} - \frac{1299}{3}\right)$$

By rigid soil
$\Rightarrow C = .107$
$\frac{.107}{.125} = .856$

$= 0.105$ m. $\Rightarrow T = .063 \times \sqrt{105} = 0.65$ Secs.

Consider Rotation of Building under EQ

Calc Δ from max^m torsional shear in walls A and D = 831 kN.

Stiffness = 3633 $\frac{kN}{m}$ $P = k\Delta$.

for $P = 831$ kN $\delta = \frac{831}{3633} = 0.228$ m.

This is not strictly correct as load is for Δ loading $\Rightarrow \delta = \frac{11}{60}\frac{wL^3}{EI}$ (v/s $\frac{PL^3}{3EI}$)

Reduce δ by $\frac{11}{60} / \frac{1}{3} = \frac{.18}{.33} = 0.56$. ← Predominant flexural deflection

Deflection due to Torsion = $.56 \times .228$
= .127 m.

for $L = 26.4 \Rightarrow .0048\,L$.

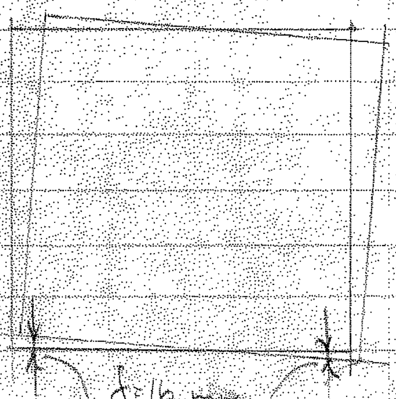
over 8 floors Δ/floor = .127/8 = .016 m

Rotation = $\frac{.016}{21.6m} = 7.4 \times 10^{-4}$

Allowable deflection = $\frac{5}{6}$ 0.1 h

δallow = $\frac{5}{6} \times 0.01 \times 3200$

= 27 mm.



$\nu = \frac{2}{Sm} = \frac{2}{1.0 \times 0.8} = 2.5$

$\nu\delta = 16 \times 2.5 = 40$mm due to torsion only.

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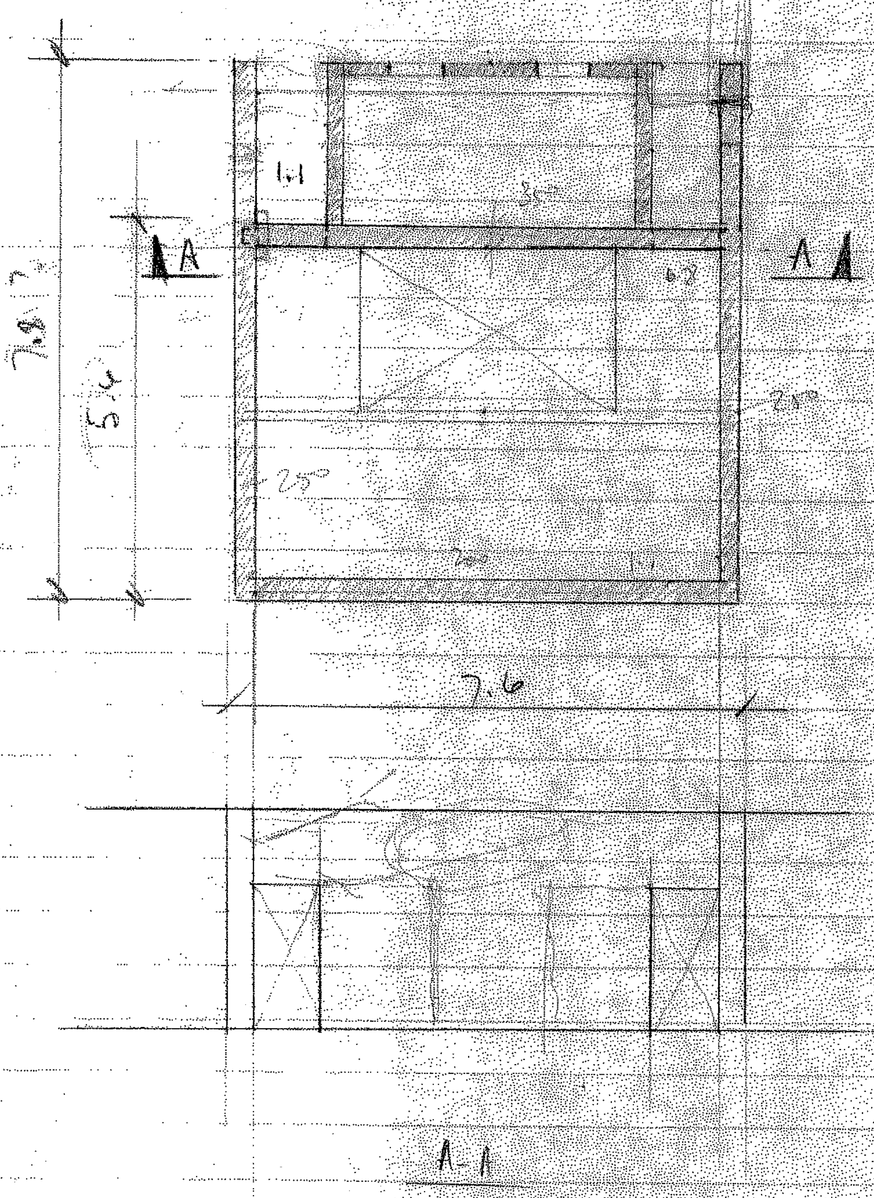
Alternative core layout with perforated shear wall at rear of lifts



$$S = \frac{3 \times 0.8 + 1 \times 1.0}{4} = 0.85$$

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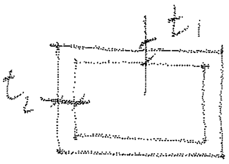
Consider the effect of the core "tube"

Essentially 5.6×7.6 m
$(a \times b)$

Torsion constant $J = \dfrac{2a^2b^2}{\left(\dfrac{a}{t_1} + \dfrac{b}{t_2}\right)}$



Assume $t_1 = t_2 = 0.3$ m

$$J = \frac{2 \times 5.6^2 \times 7.6^2}{\left(\dfrac{5.6 + 7.6}{0.3}\right)} = 82.3$$

Maximum stress factor $C_s = \dfrac{1}{2abt}$
$= (2 \times 5.6 \times 7.6 \times 0.3)^{-1}$
$= 0.039$

Ultimate torque factor $C_u = 2abt$
$= 25.5$

For $T_u = 29510$ kN-m
$\tau = 29510 / 25.5$
$= 1157$ kN/m^2
$= 1.16$ MPa

$G = 0.4 \times 25$ GPa
$= 10$ GPa

$\dfrac{kN\text{-}m}{m^4 \cdot kN}$

$$\theta = \frac{T}{GJ} = \frac{29510 \text{ kN-m}}{10\text{ GPa} \times 82} = 3.59 \times 10^{-5} \ \underline{\text{rad}/\text{m}}$$

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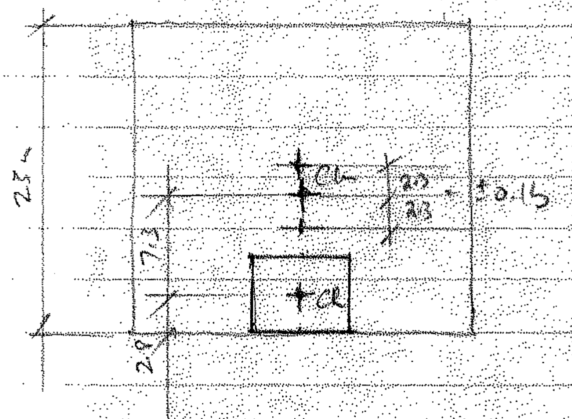
Revise design eccentricities in X direction

Assume CR in centre of "tube"

$\Rightarrow$ e_s = 10.1 m − 2.8 = 7.3 m

$\Rightarrow$ max e_{dy} = 7.3 + 2.3 = 9.6 m
min = 7.3 − 2.3 = 5.0 m.



Now
T_x max due to e_{dy} max = 3074 × 9.6 = 29510 kNm
T_x min " " e_{dy} min = " × 5.0 = 15370 kNm } X Eq

Check twist $\boxed{\theta = 3.59 \times 10^{-5} \text{ rad/m}}$ Maximum rate of twist

Over a 3.2 m storey ⇒ 3.2 × 3.59 = 1.15 × 10^{-4} rad   worst storey @ bottom.

Deflection at corner = θr = 1.15×10^{-4} × 23000
= 2.65 mm

Over 8 storeys ⇒ 8 × 2.65 = 21 mm ok.

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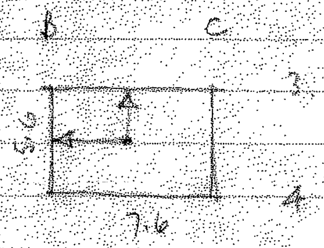
Approx check on torsional shear

$k = \frac{3EI}{L^3}$



$I_{B,C} = .3 \times \frac{5.6^3}{12} = 4.39$

$I_{3,4} = .3 \times \frac{7.6^3}{12} = 11.00$

$k \propto I$ as flexure dominant.

$T = 29500$ kNm

Element	k	r	kr	kr^2	$\frac{kr}{\Sigma kr^2}$	V_T
B	4.39	3.7	16.24	60.1	.0579	1707
C	4.39	3.7	16.24	60.1	.0579	1707
3	11.0	2.7	29.7	80.2	.106	3127
4	11.0	2.7	29.7	80.2	.106	3127
				280.6		

Check Eq'n $M = 1707 \times 7.4 + 3127 \times 5.4$
$= 29520$ kNm ok.

Shear stress Wall $_{B,C}$ $= 1707 \times 10^3 / 300 \times 5400$
$= 1.05$ MPa

Wall $_{3,4}$ $= 3127 \times 10^3 / 300 \times 7400$
$= 1.41$ MPa

$v_i = 1.16$ MPa page 23 ok.

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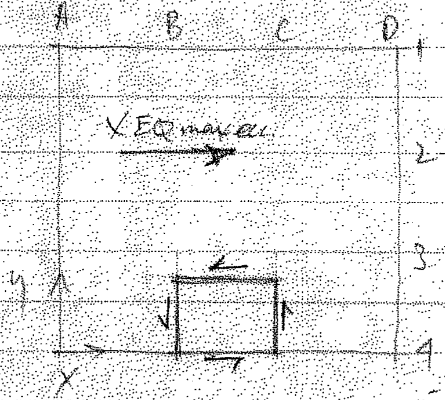
Consider Direct shear + Torsion

In other direction have 2 walls of equal stiffness ⇒ ½ direct shear per wall.

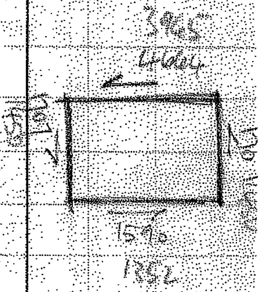
V_E = 3074 kN

$\frac{V_E}{2}$ = 1537 kN



X EQ e_{max}

Wall	Direct Shear	Torsional Shear	Σ	x .85
B	–	–1707	–1707	–1450
C	–	1707	1707	1450
3	–1537	–3127	–4664	3965
4	–1537	+ 3127	1590	1352
			3074	



X EQ e_{min}

	Direct	Torsion	Σ
B		–890	–890
C		890	890
3	–1537	–1630	–3167
4	–1537	1630	93
			3074

3965   8723

V_D Wall ③ = 2.2 x 4664 = 10260   Area reqd = 10260×10^3 / 3.6

Wall ④ = 2.2 x 1590 = 3498

8723×10^3 / 7200 x 350 = 3.5 MPa OK 350 wall.

= 2850000

÷ 7600 = 375

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Wall thickness sized on $V_{i\,max} = 3.6$ MPa

Wall (3) $V_o = 0.85 \times 10260 = 8720$ kN

Reduction due to coupled shear walls

(4) $V_o = 0.85 \times 1590 = 1351$ kN

Take wall length = 7200 mm

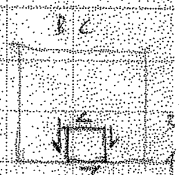
Thickness reqd Wall (3) $= 8720 \times 10^3 / 3.6 \times 7200 = 336$ — 350 OK

(4) $= 1350 \times 10^3 /$ " " $= 52$ — 200 OK

Direct + Torsional shear y EQ

$T_y = 7500$ kN-m (·25 × EQ...)

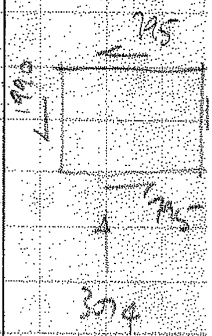
Wall	Direct Shear	Torsional Shear	Σ
B	−1537	−453	−1990
C	−1537	+453	−1084
3	0	−795	−795
4	0	+795	795



$V_b = 2.25 \times 1990 = 4478$ kN   Check

@ 3.6 MPa $t_{reqd} = 4480 \times 10^3 / 3.6 \times 5600$
222 mm

Say 250.



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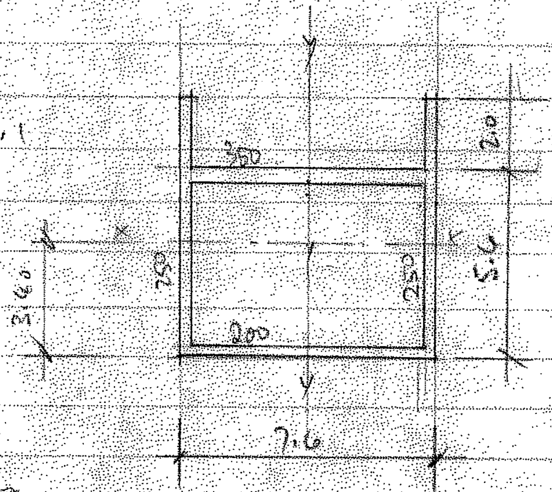
Check Deflection of Core under direct load

Area = 7.6 x 0.25 x 2
+ .35 x 7.1 + .2 x 7.1
= 7.705 m²

$\bar{x}$ = 7.6 / 2
= 3.8 m



Calc $\bar{y}$ about bottom
$$= \left(\frac{7.1^2}{2} \times .25 \times 2 + \frac{.2^2}{2} \times 7.1 + .35 \times 7.1 \times 5.425 \right) \Big/ 7.705$$

= 3.40 m

I_{xx} = $\frac{7.6^3}{12} \times .25 \times 2 + 7.6 \times .25 \times 0.4^2$ = 18.6 m⁴
+ $7.1 \times .35 \times 2.025^2$ = 10.2
+ $7.1 \times .2 \times 3.3^2$ = 15.5
44.3 m⁴

I_{yy} = $\frac{7.6^4}{12} - \frac{7.05 \times 7.1^3}{12}$ = 67.7 m⁴

Under a direct ∇ load of 3074 kN

X EQ Δx = 11 x 3074 x 26.4³ / 60 x 25 E6 x 67.7
= 0.006 m.

Y EQ Δy = 0.006 x 67.7 / 44.3
= 0.009 mm

Deflection due to flexure

CALCULATIONS

Consider the Shear Deflection

$\Delta shear = \frac{1.2\ PL}{GA}$ for point load

Approx 3074 kN @ $\frac{2}{3}$ h = 17.7 m.

$A_{xx} = 2 \times 7.6 \times .25 = 3.8\ m^2$

$A_{yy} = 7.6 \times (.35 + .2) = 4.18\ m^2$

x EQ

$\Delta shear = \frac{1.2 \times 3074 \times 17.7}{10\ E6 \times 4.18} = .0016\ m.$

y EQ

$\Delta shear = .0016 \times \frac{4.2}{3.8} = .0017\ m.$

Total deflection direct

x EQ = .006 + .0016 = .008 m

y EQ = .009 + .0018 = .011 m

Increase by $\nu_x = \frac{2}{Sm} = \frac{2}{.85 \times .8} = 2.94$

$\nu_y = \frac{2}{.8 \times 1.0} = 2.5$

$\Rightarrow \Delta E_x = .008 \times 3 = .024$ m

3-3.5 mm / floor

$\Delta E_y = .011 \times 2.5 = .028$ m

.0006 h = .0006 × 3200 = 1.92 mm > 3

CALCULATIONS

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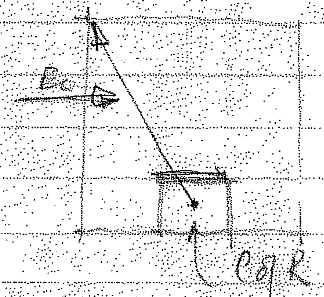
Combine Direct + torsional Deflections

At extreme corner approx 23 m from C of R

$\theta = 1.15 \times 10^{-4}$ rad - worst storey



$\Rightarrow \Delta_\theta = 1.15 \times 10^{-4} \times 23000$ at corner
$= 2.7$ mm.

Apply $\nu = 3$
$\Rightarrow \Delta_{\theta\nu} = $ (8 mm bottom storey)

Direct + torsion $(\Delta_\theta + \Delta_s)\nu = 8 + 3.5$
$\doteq 12$ mm — at corner. bottom storey

Maximum allowable $= 0.01 h \times \frac{5}{6}$

$= .01 \times 3200 \times 5/6$
$= 27$ mm $>$ 12 mm ok

Total deflection at top of building

take average rotation $= 2/3$ max^{m}
$= 2/3 \times 3.59 \times 10^{-5}$ rad/m
$= 2.4 \times 10^{-5}$ — /m

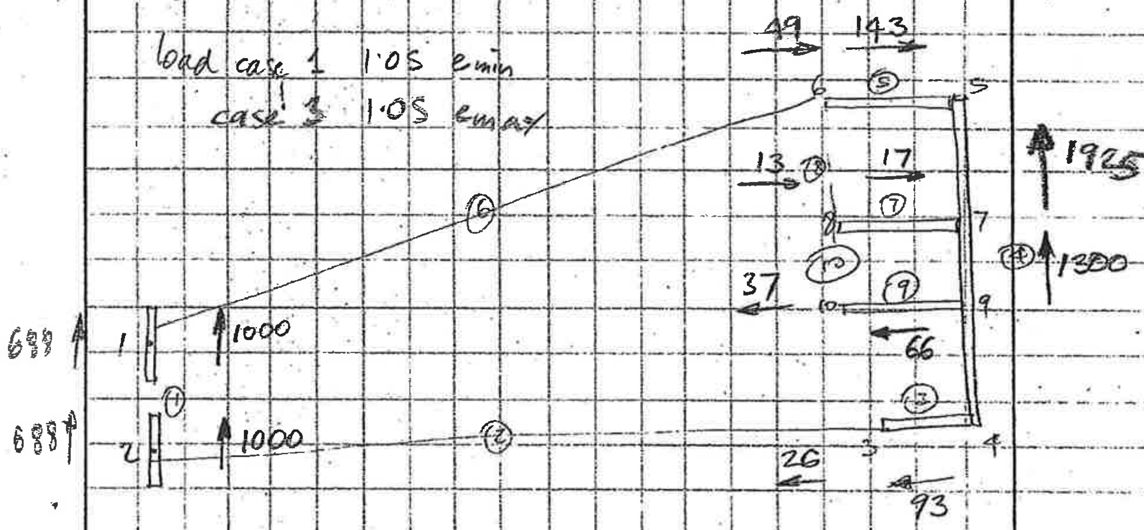
over 26.4 m $\Rightarrow \Sigma\theta = 26.4 \times 2.4 \times 10^{-5}$
$= 6.35$ E-4 rad.

At corner of bldg $r = 23000$

$\Delta_\theta = 23 \times 10^3 \times 6.35 \times 10^{-4} = 14.5$ mm
$\nu\Delta_\theta = 3 \times 14.5 = 43.5$ mm
Total deflection $\nu(\Delta_\theta + \Delta_s) = 44 + 24 = 68$ mm total

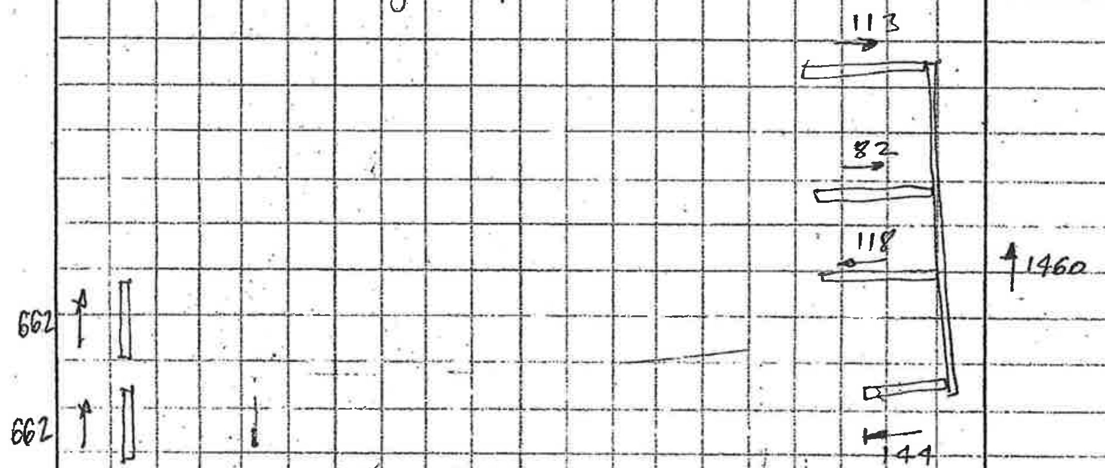
Yeq Ground Floor shears

load case 1 1·05 emin
case 3 1·05 emax



$\Sigma Y = 3300$ $\Sigma X = 1$
$\Sigma Y = 3301$ $\Sigma X = 1$
cf $V = 3300$

load case 5 dynamic, $e = e_{max}$



$\Sigma Y = 2784$

100mm (A4 print at Scale 1:1)

BUI.MAD249.0405.1

CTV Building
249 Madras Street, Christchurch

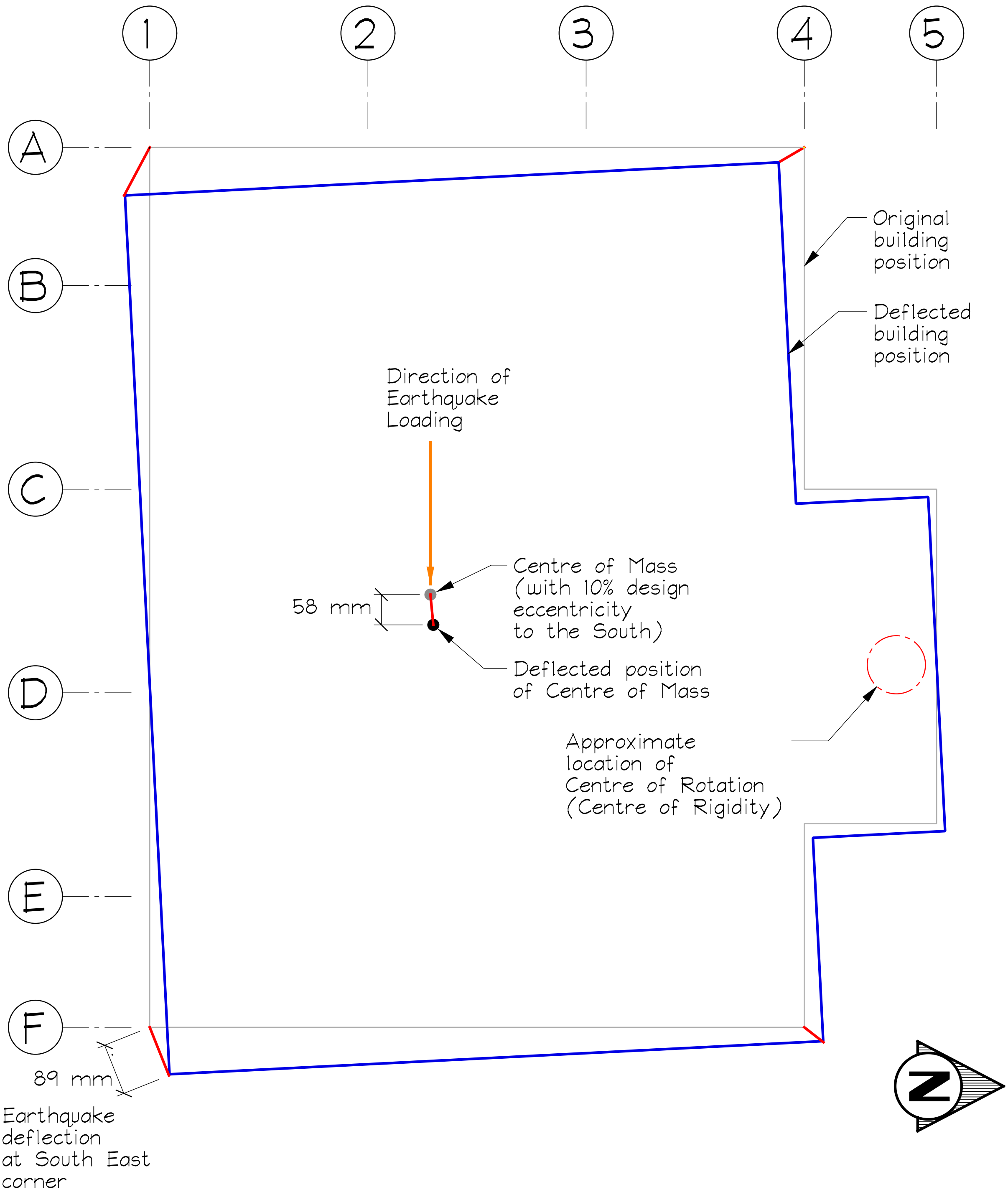


DIAGRAM OF EARTHQUAKE DEFLECTIONS
EAST-WEST LOADING
(With 10% Design Eccentricity to the South)
FOR DBH STATIC LOAD CASE
AT THE TOP FLOOR LEVEL
(SM=0.8, K/SM=2.5)

150 mm (A4 print at Scale 1:1)

Unit 4, 502 Wairakei Road, PO Box 4597, Christchurch 8140, Phone: 03 379-4014, Fax: 03 365-2449, info@eliotsinclair.co.nz www.eliotsinclair.co.nz

Plotted May20 2012 at 10:55a.m. from G:\Jobs\35\352896\Ustn\352896 M1_01_EW_EQ induced displacement_adjusted_b.dgn, at 1:1 by nms

Check building Period:

T_x = 1.06 seconds. ⇒ C = 0.089
T_y = 1.06 seconds
c.f assumed
$T_x \leq$ 0.7 sec ⇒ C = 0.125.

ie case A & B loads reduced by $\frac{0.089}{0.125}$ = factor 0.712

Building Deflections: X direction EQ.

level	load case A	load case B	max inter-storey deflection		interstorey ×1.78 $= \frac{K}{SM} \times .712$	
6	70.6	70.1	~~21.7~~	21.9	39.0	< 43.3
5	48.7	48.4	13.2		23.4	< 27
4	35.5	35.2	~~12.3~~	12.4	22.0	< 27
3	23.1	22.9	~~10.6~~	10.8	19.2	< 27
2	12.3	12.3	8.0		14.2	< 27
1	4.3	4.3	4.3		7.7	< 32.5

$\frac{K}{SM} = \frac{2.0}{1.0 \times 0.8} = 2.5 \times 0.712 = 1.78$

allow defl = $\frac{5}{6} \times 0.010 \times h$ = .0083 h

for levels 2 to 5 = 0.0083 × 3240 = 27.0 mm
6 = .0083 × 5200 = 43.3
1 3900 = 32.5

Building separation
from N & S boundaries.
either = 1.718 × 70.6 mm × 1.5 = 182 mm
or = 0.002 × 22,060 = 44 mm

CALCULATIONS

ALAN M. REAY
CONSULTING ENGINEER
CHRISTCHURCH

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Building Deflections: Y direction EQ

level	load case A	load case B	max inter-storey defl.	interstorey ×1·78.	
6	15·5	11·7	4·4	7·8	< 43·3
5	11·1	8·3	2·7	4·8	
4	8·4	6·3	2·7	4·8	< 27
3	5·7	4·3	2·3	4·1	
2	3·4	2·6	2·0	3·6	
1	1·4	1·1	1·4	2·5	< 32·5

Building separation

From E(W) boundaries

= ·002 × 22060 = 44 mm

or = 1·718 × 15·5 × 1·5 = 40 mm

∴ use 50 mm min.

WIT.HENRY.0001.65

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Mode	Period	Description	UX	UY	UZ	SumUX	SumUY	SumUZ	RX	RY	RZ	SumRX	SumRY	SumRZ
1	1.2191	N/S 1st mode	72.2225	3.4653	0	72.2225	3.4653	0	4.2791	92.0752	2.3564	4.2791	92.0752	2.3564
2	1.025673	E/W South Wall 1st mode	6.1802	47.2984	0	78.4027	50.7638	0	58.9802	7.6394	21.3723	63.2593	99.7146	23.7287
3	0.3159443	E/W North Core 1st mode	0.0484	25.6633	0	78.4511	76.4270	0	36.0456	0.0512	53.6060	99.3049	99.7659	77.3347
4	0.1883747	E/W South Wall 2nd mode	2.6730	10.6468	0	81.1241	87.0738	0	0.5592	0.0328	5.8037	99.8641	99.7986	83.1384
5	0.148953	N/S 2nd Mode	14.0106	2.5220	0	95.1347	89.5957	0	0.0760	0.1741	0.6920	99.9402	99.9727	83.8304
6	0.08060526	E/W South Wall 3rd mode	0.5526	1.7769	0	95.6873	91.3727	0	0.0145	0.0049	2.2028	99.9546	99.9776	86.0332
7	0.06188745	N/S 3rd mode	2.5764	0.7432	0	98.2638	92.1159	0	0.0155	0.0182	0.0939	99.9701	99.9958	86.1271
8	0.05598493	E/W North Core 2nd mode	0.3664	5.7055	0	98.6301	97.8214	0	0.0242	0.0016	8.3912	99.9943	99.9973	94.5182
9	0.05081683	E/W South Wall 4th mode	0.1383	0.0102	0	98.7684	97.8316	0	0.0011	0.0002	2.3682	99.9954	99.9976	96.8864
10	0.04148619	N/S North Core Col D/E4 1st mode	0.3275	0.8755	0	99.0959	98.7071	0	0.0002	0.0002	0.1576	99.9956	99.9977	97.0440
11	0.0393983	E/W South Wall 5th Mode	0.2351	0.0001	0	99.3311	98.7072	0	0.0006	0.0002	0.1668	99.9962	99.9979	97.2108
12	0.03503284	Rotation North Core 1st mode	0.4046	0.1445	0	99.7357	98.8517	0	0.0000	0.0015	0.7135	99.9962	99.9994	97.9243
13	0.02905703	N/S North Core Col D/E4 2nd mode	0.1575	0.2698	0	99.8932	99.1215	0	0.0012	0.0005	0.2727	99.9974	99.9999	98.1970
14	0.02611202	Rotation North Core 2nd mode	0.0614	0.5128	0	99.9546	99.6343	0	0.0023	0.0000	1.1285	99.9997	99.9999	99.3255
15	0.02411116		0.0387	0.0374	0	99.9933	99.6717	0	0.0000	0.0001	0.0322	99.9998	100.0000	99.3578
16	0.0195451		0.0042	0.2552	0	99.9975	99.9269	0	0.0000	0.0000	0.4978	99.9998	100.0000	99.8556
17	0.0164407		0.0021	0.0590	0	99.9996	99.9859	0	0.0002	0.0000	0.1165	100.0000	100.0000	99.9721
18	0.01465746		0.0004	0.0141	0	100.0000	100.0000	0	0.0000	0.0000	0.0279	100.0000	100.0000	100.0000
19	0.01066402		0.0000	0.0000	0	100.0000	100.0000	0	0.0000	0.0000	0.0000	100.0000	100.0000	100.0000
20	0.009208754		0.0000	0.0000	0	100.0000	100.0000	0	0.0000	0.0000	0.0000	100.0000	100.0000	100.0000
21	0.005494458		0.0000	0.0000	0	100.0000	100.0000	0	0.0000	0.0000	0.0000	100.0000	100.0000	100.0000

Centre of Mass ERSA Inter-storey East-West Drifts

10% eccentricty of mass south and east of centre. ERSA drifts are from differences in storey displacements and may not be maximum drifts.
No allowance has been made for inelastic effects.

		No Line A Masonry Infill			No Line A Masonry Infill			
Level	**Storey Heights m**	**Static NZS 4203:1984 displacement**	**Static NZS 4203:1984 Interstorey displacement**	**Static NZS 4203:1984 K/SM=2.5**	**ERSA NZS 4203:1984 displacement**	**ERSA NZS 4203:1984 Interstorey displacement**	**ERSA NZS 4203:1984 K/SM=2.75**	**ERSA NZS 4203:1984 ULS S=5**
North -South Earthquake								
L5 - L6	3240	0.7	0.2	0.02%	9.2	2.4	0.20%	0.37%
L4	3240	0.5	0.1	0.01%	6.8	2.1	0.18%	0.32%
L3	3240	0.4	0.2	0.02%	4.7	2.0	0.17%	0.31%
L2	3240	0.2	0.1	0.01%	2.7	1.6	0.14%	0.25%
L1	3825	0.1	0.1	0.01%	1.1	1.1	0.08%	0.14%
East-West Earthquake								
L5 - L6	3240	23.2	5.9	0.46%	26.5	6.8	0.58%	1.05%
L4	3240	17.3	5.3	0.41%	19.7	6.1	0.52%	0.94%
L3	3240	12.0	4.9	0.38%	13.6	5.7	0.48%	0.88%
L2	3240	7.1	4.1	0.32%	7.9	4.6	0.39%	0.71%
L1	3825	3.0	3.0	0.20%	3.3	3.3	0.24%	0.43%

Centre of Mass ERSA Inter-storey North-South Drifts

10% eccentricty of mass south and east of centre. ERSA drifts are from differences in storey displacements and may not be maximum drifts.
No allowance has been made for inelastic effects.

		No Line A Masonry Infill			No Line A Masonry Infill			
Level	**Storey Heights m**	**Static NZS 4203:1984 displacement**	**Static NZS 4203:1984 Interstorey displacement**	**Static NZS 4203:1984 K/SM=2.5**	**ERSA NZS 4203:1984 displacement**	**ERSA NZS 4203:1984 Interstorey displacement**	**ERSA NZS 4203:1984 K/SM=2.75**	**ERSA NZS 4203:1984 ULS S=5**
North -South Earthquake								
L5 - L6	3240	35.6	7.6	0.59%	30.9	6.6	0.56%	1.02%
L4	3240	28.0	7.6	0.59%	24.3	6.7	0.57%	1.03%
L3	3240	20.4	7.4	0.57%	17.6	6.4	0.54%	0.99%
L2	3240	13.0	6.7	0.52%	11.2	5.8	0.49%	0.90%
L1	3825	6.3	6.3	0.41%	5.4	5.4	0.39%	0.71%
East-West Earthquake								
L5 - L6	3240	2.1	0.5	0.04%	10.0	2.1	0.18%	0.32%
L4	3240	1.6	0.7	0.05%	7.9	2.0	0.17%	0.31%
L3	3240	0.9	0.5	0.04%	5.9	2.0	0.17%	0.31%
L2	3240	0.4	0.3	0.02%	3.9	2.0	0.17%	0.31%
L1	3825	0.1	0.1	0.01%	1.9	1.9	0.14%	0.25%

Centre of Mass ERSA Inter-storey Resultants Drifts

10% eccentricty of mass south and east of centre. ERSA resultant drifts shown are only resultant of maximum east-west and north-south drifts which may not occur concurrently

		No Line A Masonry Infill			No Line A Masonry Infill			
Level	**Storey Heights m**	**Static NZS 4203:1984 displacement**	**Static NZS 4203:1984 Interstorey displacement**	**Static NZS 4203:1984 K/SM=2.5**	**ERSA NZS 4203:1984 displacement**	**ERSA NZS 4203:1984 Interstorey displacement**	**ERSA NZS 4203:1984 K/SM=2.75**	**ERSA NZS 4203:1984 ULS S=5**
North -South Earthquake								
L5 - L6	3.24	35.6	7.6	0.59%	32.2	7.0	0.60%	1.08%
L4	3.24	28.0	7.6	0.59%	25.2	7.0	0.60%	1.08%
L3	3.24	20.4	7.4	0.57%	18.2	6.7	0.57%	1.03%
L2	3.24	13.0	6.7	0.52%	11.5	6.0	0.51%	0.93%
L1	3.825	6.3	6.3	0.41%	5.5	5.5	0.40%	0.72%
East-West Earthquake								
L5 - L6	3.24	23.3	5.9	0.46%	28.3	7.1	0.60%	1.10%
L4	3.24	17.4	5.3	0.41%	21.2	6.4	0.54%	0.99%
L3	3.24	12.0	4.9	0.38%	14.8	6.0	0.51%	0.93%
L2	3.24	7.1	4.1	0.32%	8.8	5.0	0.43%	0.77%
L1	3.825	3.0	3.0	0.20%	3.8	3.8	0.27%	0.50%

A/1 ERSA Inter-storey East-West Drifts

10% eccentricty of mass south and east of centre. ERSA drifts are point drifts from ETABS. No allowance has been made for inelastic effects.

Level	Storey Heights m	No Line A Masonry Infill Static NZS 4203:1984 displacement	Static NZS 4203:1984 Interstorey displacement	Static NZS 4203:1984 K/SM=2.5	No Line A Masonry Infill ERSA NZS 4203:1984 Interstorey displacement	ERSA NZS 4203:1984 K/SM=2.75	ERSA NZS 4203:1984 ULS S=5
North -South Earthquake							
L5 - L6	3240	0.9	0.3	0.02%	3.6	0.30%	0.55%
L4	3240	0.6	0.2	0.02%	3.2	0.28%	0.50%
L3	3240	0.4	0.1	0.01%	3.2	0.28%	0.50%
L2	3240	0.3	0.2	0.02%	2.6	0.22%	0.40%
L1	3825	0.1	0.1	0.01%	1.9	0.14%	0.25%
East-West Earthquake							
L5 - L6	3240	33.1	8.0	0.62%	9.7	0.83%	1.50%
L4	3240	25.1	7.9	0.61%	9.4	0.80%	1.45%
L3	3240	17.2	7.2	0.56%	8.7	0.74%	1.35%
L2	3240	10.0	5.9	0.46%	7.1	0.61%	1.10%
L1	3825	4.1	4.1	0.27%	5.0	0.36%	0.65%

A/1 ERSA Inter-storey North-South Drifts

10% eccentricty of mass south and east of centre. ERSA drifts are point drifts from ETABS. No allowance has been made for inelastic effects.

Level	Storey Heights m	No Line A Masonry Infill Static NZS 4203:1984 displacement	Static NZS 4203:1984 Interstorey displacement	Static NZS 4203:1984 K/SM=2.5	No Line A Masonry Infill ERSA NZS 4203:1984 Interstorey displacement	ERSA NZS 4203:1984 K/SM=2.75	ERSA NZS 4203:1984 ULS S=5
North -South Earthquake							
L5 - L6	3240	35.8	7.7	0.59%	5.5	0.47%	0.85%
L4	3240	28.1	7.6	0.59%	5.2	0.44%	0.80%
L3	3240	20.5	7.4	0.57%	5.2	0.44%	0.80%
L2	3240	13.1	6.8	0.52%	4.9	0.41%	0.75%
L1	3825	6.3	6.3	0.41%	4.6	0.33%	0.60%
East-West Earthquake							
L5 - L6	3240	17.6	4.4	0.34%	7.5	0.63%	1.15%
L4	3240	13.2	4.2	0.32%	7.5	0.63%	1.15%
L3	3240	9.0	3.9	0.30%	6.8	0.58%	1.05%
L2	3240	5.1	3.1	0.24%	5.8	0.50%	0.90%
L1	3825	2.0	2.0	0.13%	4.6	0.33%	0.60%

A/1 ERSA Inter-storey Resultants Drifts

10% eccentricty of mass south and east of centre. ERSA resultant drifts shown are only resultant of maximum east-west and north-south drifts which may not occur concurrently

Level	Storey Heights m	No Line A Masonry Infill Static NZS 4203:1984 displacement	Static NZS 4203:1984 Interstorey displacement	Static NZS 4203:1984 K/SM=2.5	No Line A Masonry Infill ERSA NZS 4203:1984 Interstorey displacement	ERSA NZS 4203:1984 K/SM=2.75	ERSA NZS 4203:1984 ULS S=5
North -South Earthquake							
L5 - L6	3.24	35.8	7.7	0.59%	6.6	0.56%	1.01%
L4	3.24	28.1	7.6	0.59%	6.1	0.52%	0.94%
L3	3.24	20.5	7.4	0.57%	6.1	0.52%	0.94%
L2	3.24	13.1	6.8	0.52%	5.5	0.47%	0.85%
L1	3.825	6.3	6.3	0.41%	5.0	0.36%	0.65%
East-West Earthquake							
L5 - L6	3.24	37.5	9.1	0.70%	12.2	1.04%	1.89%
L4	3.24	28.4	8.9	0.69%	12.0	1.02%	1.85%
L3	3.24	19.4	8.2	0.63%	11.1	0.94%	1.71%
L2	3.24	11.2	6.7	0.51%	9.2	0.78%	1.42%
L1	3.825	4.6	4.6	0.30%	6.8	0.49%	0.88%

A/4 ERSA Inter-storey East-West Drifts

10% eccentricty of mass south and east of centre. ERSA drifts are point drifts from ETABS. No allowance has been made for inelastic effects.

		No Line A Masonry Infill			No Line A Masonry Infill		
Level	**Storey Heights m**	**Static NZS 4203:1984 displacement**	**Static NZS 4203:1984 Interstorey displacement**	**Static NZS 4203:1984 K/SM=2.5**	**ERSA NZS 4203:1984 Interstorey displacement**	**ERSA NZS 4203:1984 K/SM=2.75**	**ERSA NZS 4203:1984 ULS S=5**
North -South Earthquake							
L5 - L6	3240	0.6	0.1	0.01%	0.6	0.06%	0.10%
L4	3240	0.5	0.2	0.02%	0.6	0.06%	0.10%
L3	3240	0.3	0.1	0.01%	0.6	0.06%	0.10%
L2	3240	0.2	0.1	0.01%	0.3	0.03%	0.05%
L1	3825	0.1	0.1	0.01%	0.4	0.03%	0.05%
East-West Earthquake							
L5 - L6	3240	10.3	2.4	0.19%	2.3	0.19%	0.35%
L4	3240	7.9	2.3	0.18%	2.3	0.19%	0.35%
L3	3240	5.6	2.2	0.17%	1.9	0.17%	0.30%
L2	3240	3.4	1.8	0.14%	1.6	0.14%	0.25%
L1	3825	1.6	1.6	0.10%	1.5	0.11%	0.20%

A/4 ERSA Inter-storey North-South Drifts

10% eccentricty of mass south and east of centre. ERSA drifts are point drifts from ETABS. No allowance has been made for inelastic effects.

		No Line A Masonry Infill			No Line A Masonry Infill		
Level	**Storey Heights m**	**Static NZS 4203:1984 displacement**	**Static NZS 4203:1984 Interstorey displacement**	**Static NZS 4203:1984 K/SM=2.5**	**ERSA NZS 4203:1984 Interstorey displacement**	**ERSA NZS 4203:1984 K/SM=2.75**	**ERSA NZS 4203:1984 ULS S=5**
North -South Earthquake							
L5 - L6	3240	35.8	7.7	0.59%	5.5	0.47%	0.85%
L4	3240	28.1	7.6	0.59%	5.2	0.44%	0.80%
L3	3240	20.5	7.4	0.57%	5.2	0.44%	0.80%
L2	3240	13.1	6.8	0.52%	4.9	0.41%	0.75%
L1	3825	6.3	6.3	0.41%	4.6	0.33%	0.60%
East-West Earthquake							
L5 - L6	3240	17.6	4.4	0.34%	7.5	0.63%	1.15%
L4	3240	13.2	4.2	0.32%	7.5	0.63%	1.15%
L3	3240	9.0	3.9	0.30%	6.8	0.58%	1.05%
L2	3240	5.1	3.1	0.24%	5.8	0.50%	0.90%
L1	3825	2.0	2.0	0.13%	4.6	0.33%	0.60%

A/4 ERSA Inter-storey Resultants Drifts

10% eccentricty of mass south and east of centre. ERSA resultant drifts shown are only resultant of maximum east-west and north-south drifts which may not occur concurrently

		No Line A Masonry Infill			No Line A Masonry Infill		
Level	**Storey Heights m**	**Static NZS 4203:1984 displacement**	**Static NZS 4203:1984 Interstorey displacement**	**Static NZS 4203:1984 K/SM=2.5**	**ERSA NZS 4203:1984 Interstorey displacement**	**ERSA NZS 4203:1984 K/SM=2.75**	**ERSA NZS 4203:1984 ULS S=5**
North -South Earthquake							
L5 - L6	3.24	35.8	7.7	0.59%	5.5	0.47%	0.86%
L4	3.24	28.1	7.6	0.59%	5.2	0.44%	0.81%
L3	3.24	20.5	7.4	0.57%	5.2	0.44%	0.81%
L2	3.24	13.1	6.8	0.52%	4.9	0.41%	0.75%
L1	3.825	6.3	6.3	0.41%	4.6	0.33%	0.60%
East-West Earthquake							
L5 - L6	3.24	20.4	5.0	0.39%	7.8	0.66%	1.20%
L4	3.24	15.4	4.8	0.37%	7.8	0.66%	1.20%
L3	3.24	10.6	4.5	0.35%	7.1	0.60%	1.09%
L2	3.24	6.1	3.6	0.28%	6.1	0.51%	0.93%
L1	3.825	2.6	2.6	0.17%	4.8	0.35%	0.63%

F/1 ERSA Inter-storey East-West Drifts

10% eccentricty of mass south and east of centre. ERSA drifts are point drifts from ETABS. No allowance has been made for inelastic effects.

		No Line A Masonry Infill			No Line A Masonry Infill		
Level	Storey Heights m	Static NZS 4203:1984 displacement	Static NZS 4203:1984 Interstorey displacement	Static NZS 4203:1984 K/SM=2.5	ERSA NZS 4203:1984 Interstorey displacement	ERSA NZS 4203:1984 K/SM=2.75	ERSA NZS 4203:1984 ULS S=5
North -South Earthquake							
L5 - L6	3240	0.9	0.3	0.02%	3.6	0.30%	0.55%
L4	3240	0.6	0.2	0.02%	3.2	0.28%	0.50%
L3	3240	0.4	0.1	0.01%	3.2	0.28%	0.50%
L2	3240	0.3	0.2	0.02%	2.6	0.22%	0.40%
L1	3825	0.1	0.1	0.01%	1.9	0.14%	0.25%
East-West Earthquake							
L5 - L6	3240	33.1	8.0	0.62%	9.7	0.83%	1.50%
L4	3240	25.1	7.9	0.61%	9.4	0.80%	1.45%
L3	3240	17.2	7.2	0.56%	8.7	0.74%	1.35%
L2	3240	10.0	5.9	0.46%	7.1	0.61%	1.10%
L1	3825	4.1	4.1	0.27%	5.0	0.36%	0.65%

F/1 ERSA Inter-storey North-South Drifts

10% eccentricty of mass south and east of centre. ERSA drifts are point drifts from ETABS. No allowance has been made for inelastic effects.

		No Line A Masonry Infill			No Line A Masonry Infill		
Level	Storey Heights m	Static NZS 4203:1984 displacement	Static NZS 4203:1984 Interstorey displacement	Static NZS 4203:1984 K/SM=2.5	ERSA NZS 4203:1984 Interstorey displacement	ERSA NZS 4203:1984 K/SM=2.75	ERSA NZS 4203:1984 ULS S=5
North -South Earthquake							
L5 - L6	3240	35.4	7.5	0.58%	7.5	0.63%	1.15%
L4	3240	27.9	7.6	0.59%	7.5	0.63%	1.15%
L3	3240	20.3	7.3	0.56%	7.5	0.63%	1.15%
L2	3240	13.0	6.8	0.52%	6.5	0.55%	1.00%
L1	3825	6.2	6.2	0.41%	5.7	0.41%	0.75%
East-West Earthquake							
L5 - L6	3240	13.2	3.3	0.25%	3.2	0.28%	0.50%
L4	3240	9.9	3.2	0.25%	3.2	0.28%	0.50%
L3	3240	6.7	3.0	0.23%	2.9	0.25%	0.45%
L2	3240	3.7	2.3	0.18%	2.6	0.22%	0.40%
L1	3825	1.4	1.4	0.09%	1.9	0.14%	0.25%

F/1 ERSA Inter-storey Resultants Drifts

10% eccentricty of mass south and east of centre. ERSA resultant drifts shown are only resultant of maximum east-west and north-south drifts which may not occur concurrently

		No Line A Masonry Infill			No Line A Masonry Infill		
Level	Storey Heights m	Static NZS 4203:1984 displacement	Static NZS 4203:1984 Interstorey displacement	Static NZS 4203:1984 K/SM=2.5	ERSA NZS 4203:1984 Interstorey displacement	ERSA NZS 4203:1984 K/SM=2.75	ERSA NZS 4203:1984 ULS S=5
North -South Earthquake							
L5 - L6	3.24	35.4	7.5	0.58%	8.3	0.70%	1.27%
L4	3.24	27.9	7.6	0.59%	8.1	0.69%	1.25%
L3	3.24	20.3	7.3	0.56%	8.1	0.69%	1.25%
L2	3.24	13.0	6.8	0.52%	7.0	0.59%	1.08%
L1	3.825	6.2	6.2	0.41%	6.0	0.43%	0.79%
East-West Earthquake							
L5 - L6	3.24	35.6	8.7	0.67%	10.2	0.87%	1.58%
L4	3.24	27.0	8.5	0.66%	9.9	0.84%	1.53%
L3	3.24	18.5	7.8	0.60%	9.2	0.78%	1.42%
L2	3.24	10.7	6.3	0.49%	7.6	0.64%	1.17%
L1	3.825	4.3	4.3	0.28%	5.3	0.38%	0.70%

F/4 ERSA Inter-storey East-West Drifts

10% eccentricty of mass south and east of centre. ERSA drifts are point drifts from ETABS. No allowance has been made for inelastic effects.

		No Line A Masonry Infill			No Line A Masonry Infill		
Level	**Storey Heights m**	**Static NZS 4203:1984 displacement**	**Static NZS 4203:1984 Interstorey displacement**	**Static NZS 4203:1984 K/SM=2.5**	**ERSA NZS 4203:1984 Interstorey displacement**	**ERSA NZS 4203:1984 K/SM=2.75**	**ERSA NZS 4203:1984 ULS S=5**
North -South Earthquake							
L5 - L6	3240	0.6	0.1	0.01%	0.6	0.06%	0.10%
L4	3240	0.5	0.2	0.02%	0.6	0.06%	0.10%
L3	3240	0.3	0.1	0.01%	0.6	0.06%	0.10%
L2	3240	0.2	0.1	0.01%	0.3	0.03%	0.05%
L1	3825	0.1	0.1	0.01%	0.4	0.03%	0.05%
East-West Earthquake							
L5 - L6	3240	10.3	2.4	0.19%	2.3	0.19%	0.35%
L4	3240	7.9	2.3	0.18%	2.3	0.19%	0.35%
L3	3240	5.6	2.2	0.17%	1.9	0.17%	0.30%
L2	3240	3.4	1.8	0.14%	1.6	0.14%	0.25%
L1	3825	1.6	1.6	0.10%	1.5	0.11%	0.20%

F/4 ERSA Inter-storey North-South Drifts

10% eccentricty of mass south and east of centre. ERSA drifts are point drifts from ETABS. No allowance has been made for inelastic effects.

		No Line A Masonry Infill			No Line A Masonry Infill		
Level	**Storey Heights m**	**Static NZS 4203:1984 displacement**	**Static NZS 4203:1984 Interstorey displacement**	**Static NZS 4203:1984 K/SM=2.5**	**ERSA NZS 4203:1984 Interstorey displacement**	**ERSA NZS 4203:1984 K/SM=2.75**	**ERSA NZS 4203:1984 ULS S=5**
North -South Earthquake							
L5 - L6	3240	35.4	7.5	0.58%	7.5	0.63%	1.15%
L4	3240	27.9	7.6	0.59%	7.5	0.63%	1.15%
L3	3240	20.3	7.3	0.56%	7.5	0.63%	1.15%
L2	3240	13.0	6.8	0.52%	6.5	0.55%	1.00%
L1	3825	6.2	6.2	0.41%	5.7	0.41%	0.75%
East-West Earthquake							
L5 - L6	3240	13.2	3.3	0.25%	3.2	0.28%	0.50%
L4	3240	9.9	3.2	0.25%	3.2	0.28%	0.50%
L3	3240	6.7	3.0	0.23%	2.9	0.25%	0.45%
L2	3240	3.7	2.3	0.18%	2.6	0.22%	0.40%
L1	3825	1.4	1.4	0.09%	1.9	0.14%	0.25%

F/4 ERSA Inter-storey Resultants Drifts

10% eccentricty of mass south and east of centre. ERSA resultant drifts shown are only resultant of maximum east-west and north-south drifts which may not occur concurrently

		No Line A Masonry Infill			No Line A Masonry Infill		
Level	**Storey Heights m**	**Static NZS 4203:1984 displacement**	**Static NZS 4203:1984 Interstorey displacement**	**Static NZS 4203:1984 K/SM=2.5**	**ERSA NZS 4203:1984 Interstorey displacement**	**ERSA NZS 4203:1984 K/SM=2.75**	**ERSA NZS 4203:1984 ULS S=5**
North -South Earthquake							
L5 - L6	3.24	35.4	7.5	0.58%	7.5	0.63%	1.15%
L4	3.24	27.9	7.6	0.59%	7.5	0.63%	1.15%
L3	3.24	20.3	7.3	0.56%	7.5	0.63%	1.15%
L2	3.24	13.0	6.8	0.52%	6.5	0.55%	1.00%
L1	3.825	6.2	6.2	0.41%	5.8	0.41%	0.75%
East-West Earthquake							
L5 - L6	3.24	16.7	4.1	0.31%	4.0	0.34%	0.61%
L4	3.24	12.7	3.9	0.30%	4.0	0.34%	0.61%
L3	3.24	8.7	3.7	0.29%	3.5	0.30%	0.54%
L2	3.24	5.0	2.9	0.23%	3.1	0.26%	0.47%
L1	3.825	2.1	2.1	0.14%	2.4	0.18%	0.32%

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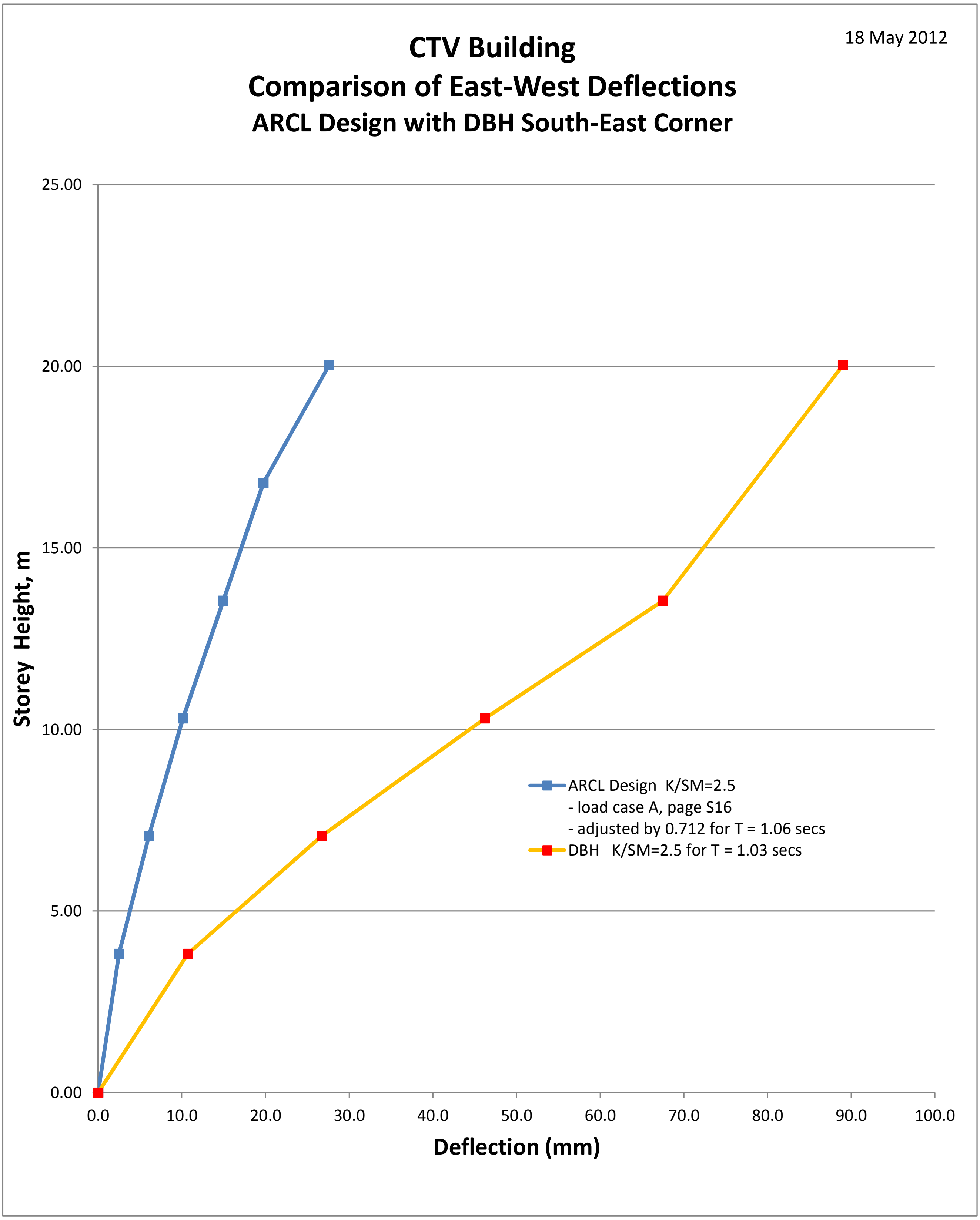
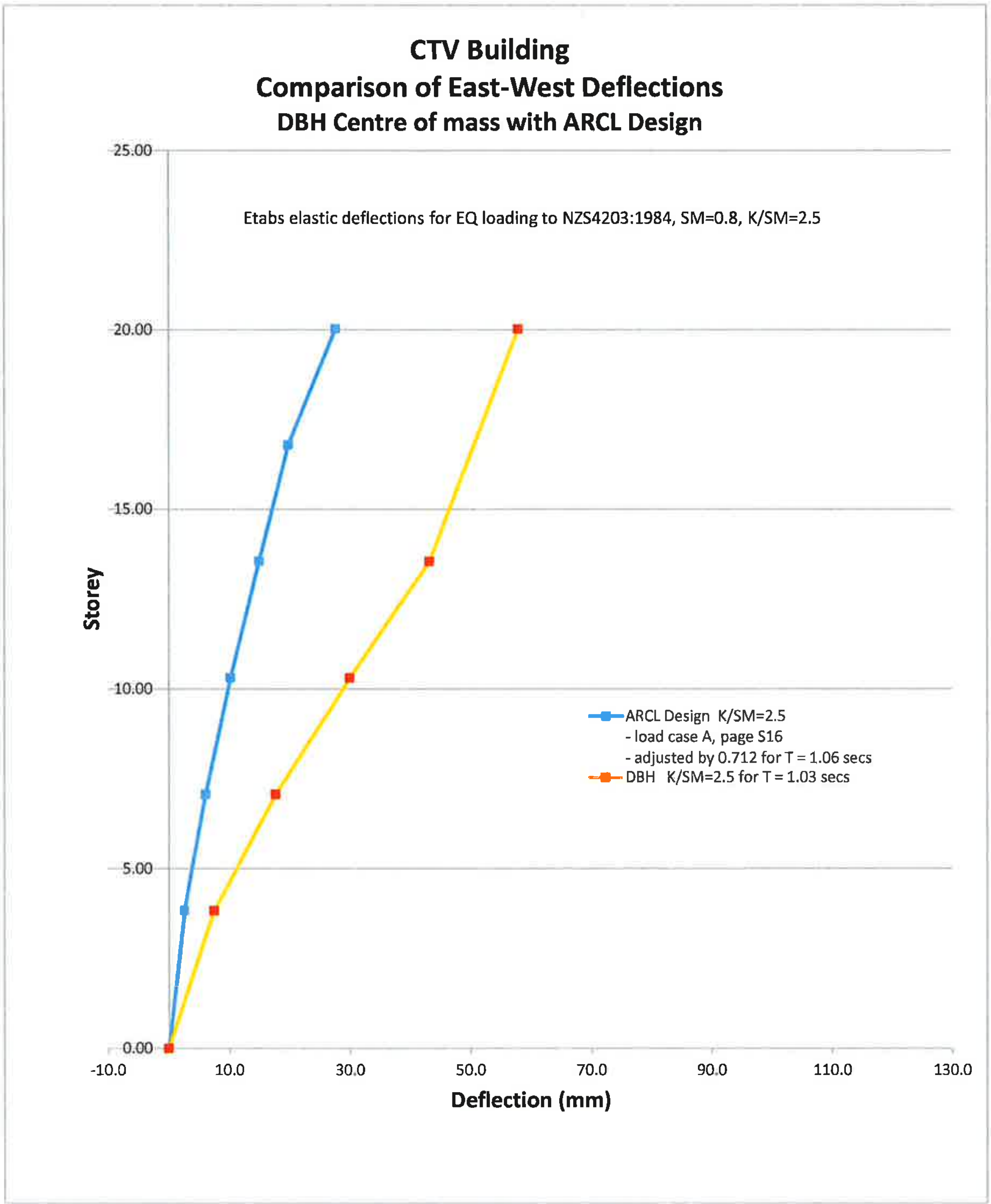
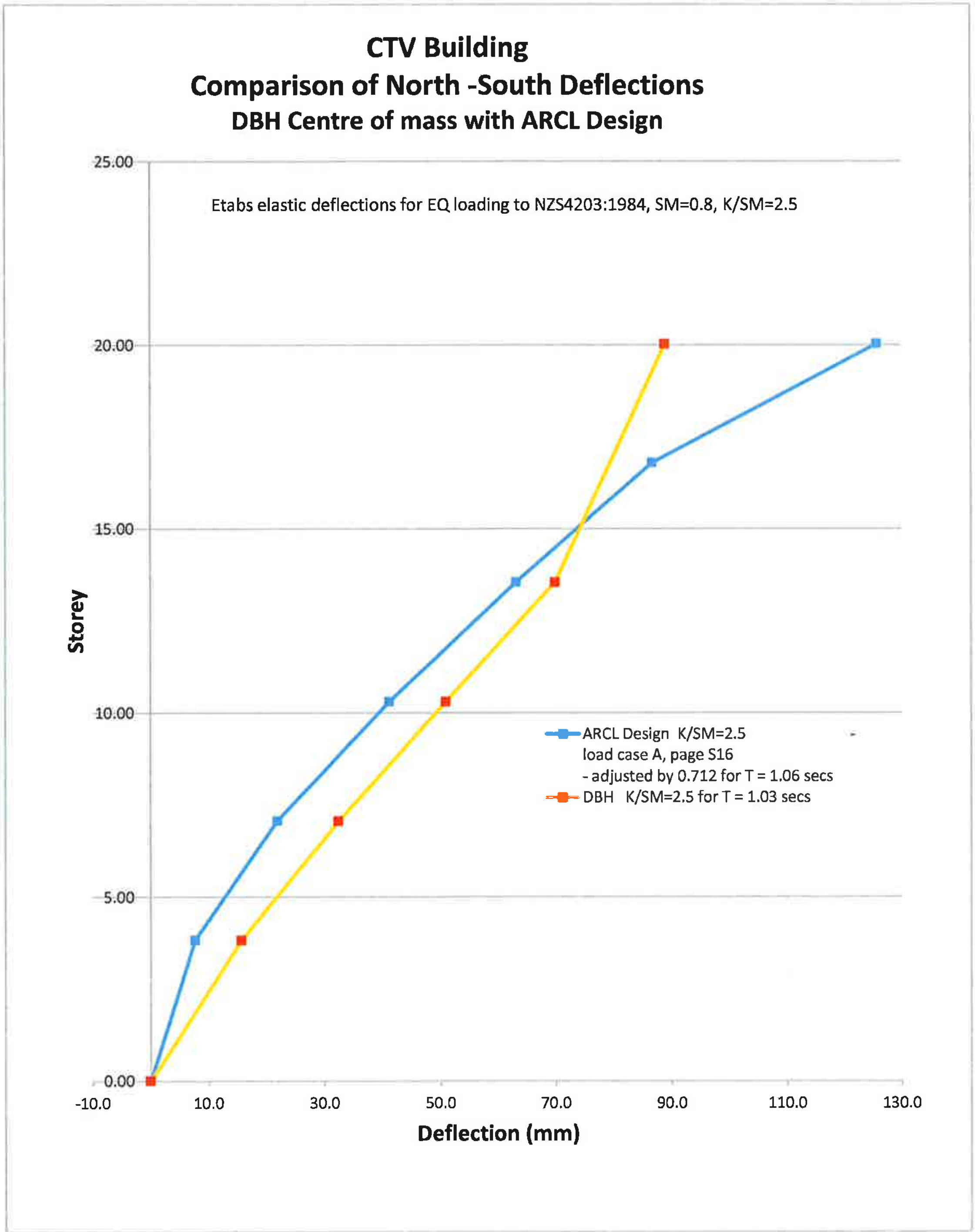



CTV Building
Comparison of East-West Deflections
DBH Centre of mass with ARCL Design
Etabs elastic deflections for EQ loading to NZS4203:1984, SM=0.8, K/SM=2.5
Storey
25.00
20.00
15.00
10.00
5.00
0.00
-10.0
10.0
30.0
50.0
70.0
90.0
110.0
130.0
Deflection (mm)
ARCL Design K/SM=2.5
- load case A, page S16
- adjusted by 0.712 for T = 1.06 secs
DBH K/SM=2.5 for T = 1.03 secs


CTV Building
Comparison of North -South Deflections
DBH Centre of mass with ARCL Design
Etabs elastic deflections for EQ loading to NZS4203:1984, SM=0.8, K/SM=2.5
Storey
25.00
20.00
15.00
10.00
5.00
0.00
-10.0
10.0
30.0
50.0
70.0
90.0
110.0
130.0
Deflection (mm)
ARCL Design K/SM=2.5
load case A, page S16
- adjusted by 0.712 for T = 1.06 secs
DBH K/SM=2.5 for T = 1.03 secs

Base shear Coefficient.

$Cd = CSmR$

$C = 0.125$ assuming $T < 0.45$ secs rigid
< 0.70 secs flexible

$m = 0.8$
$R = 1.0$ category 4
$S = 1.0$ $Z = 1.0$

Assume ductile cantilever shear walls Y Direction
$h = 26.4$ m } $\frac{h_w}{L_w} = 3.67$
$l = 7.2$ m }
$Z = 2.5 - 0.75 \times 3.67 = -.25 < 1.0$ $Sm_y = 0.8$
$Z = 1.0$

$\Rightarrow$ $Cd = 0.125 \times 0.8 = 0.10$ | $Cd_x = 0.10 \times .85 = 0.085$
$Cd_y = 0.10 = 0.10$

Base Shear = Cd Swt
= 0.10×30731
= 3073 kN.

X direction Lave coupled Shear walls
Carrying majority of Shear

Assume $S = 0.85$ initially & check $Sm_x = .68$
(assume A > .67)
$\Rightarrow$ $S = 0.8$ but increase
to 0.85 as new wall
not perforated

$\Rightarrow Sm_x = 0.8 \times .85 = 0.68$ + factor on dynamic

3.5.2.2 *Torsional effects*

3.5.2.2.1 For symmetrical or moderately unbalanced buildings for which torsional effects are calculated by the static method of clause 3.4.7 account shall be taken of not less than the first three modes for each direction under consideration.

C3.5.2.2 See clause 3.4.7.2.

3.5.2.2.2 Where dynamic torsional effects are included in the spectral modal analysis, account shall be taken of not less than four modes for each direction under consideration, two of them predominantly translational and two predominantly torsional. The model shall include the effects of accidental eccentricities of ±0.1 *b*. For moderately unbalanced buildings the torsional effect shall be not less than that calculated by the static method of clause 3.4.7.

3.5.2.3 *Shear*

3.5.2.3.1 The shear at any level shall be taken as the square root of the sum of the squares of the modal shears at that height.

C3.5.2.4 When a building is designed to resist the more accurate distribution of loads given by spectral modal analysis then an improved performance will result. Base shear values are therefore reduced to 90 percent of the values given in section 3.4.

3.5.2.4 *Scaling factor*

3.5.2.4.1 The value of the scaling factor *K* shall be chosen so that in accordance with clauses 3.5.2.1 to 3.5.2.3 inclusive the computed base shear *V* is not less than 0.9 $C_d W_t$.

C3.5.2.5 At some levels of a building the spectral modal analysis might give load values much lower than those given by the equivalent static force method. These low local values are obtained partly as a result of neglecting some of the effects of inelastic deformation on the building response, and therefore full advantage cannot be taken of the apparent local reduction in loads.

3.5.2.5 *Minimum shear values*

3.5.2.5.1 At any level the shear derived in accordance with clause 3.5.2.3 shall be taken as not less than 80 percent of the values computed by the equivalent static forces method specified in section 3.4.

3.5.2.6 *Horizontal forces and overturning moments*

3.5.2.6.1 The horizontal forces and overturning moments shall be derived from the shears given by clauses 3.5.2.1 to 3.5.2.5 inclusive.

C3.5.3 The value which should be selected for the equivalent static base shear (C_d) becomes increasingly uncertain as the fundamental period of the building increases beyond 1.5 s. For this period range the C values of fig. 3 are intended to be conservative. Such long-period buildings should have their horizontal seismic loads selected on the basis of special studies.

The following may be adopted as a guide in the selection of design earthquakes, the modal damping, and the reduction factor for ductility: If the building stiffness, or the building height, is scaled to reduce its fundamental period to 1.5 s then the procedures adopted for the dynamic analysis of the non-scaled building should result in a base shear for the scaled building which is not less than 90 percent of the equivalent static base shear; that is 0.9 $C_d W_t$.

3.5.3 **Numerical integration response analysis**

3.5.3.1 Numerical integration response analysis may be used to obtain additional information on building behaviour, particularly in the post-elastic range, to supplement that obtained by spectral modal analysis.

3.6 SPECIFIC REQUIREMENTS FOR PARTICULAR ELEMENTS

3.6.1 General

3.6.1.1 Clause 3.4.9 shall be subject to the specific requirements of this section for the particular elements covered below.

CALCULATIONS

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Scale factor for Dynamic

Xeq , Base shears from p. S11

$$K = \frac{0{\cdot}9\ V_{static}}{V_{dyn}} = \frac{0{\cdot}9 \times 3300}{2233} = 1{\cdot}33$$

But static loads must be scaled down by 0·712 to allow for building period larger than assumed (p. S15)

This reduces V_{static} & also K

$\therefore$ K = 1·33 x 0·712 = 0·95

further change to dynamic x SM = 1·0 x 0·8

$\therefore$ multiplier for dynamic = ·95 x ·8 = 0·76

Yeq , Base shears from p. S12

$$K = \frac{0{\cdot}9 \times 3300}{2784} = 1{\cdot}07$$

modified K (as above Xeq) = 1·07 x 0·712 x 0·8 = 0·61

Multiplier for Static

Compare Dynamic with 0·8 static

Multiplier for static loads derived from 1·0 S etc = 0·8 x 0·712 = 0·57

CALCULATIONS

ALAN M. REAY
CONSULTING ENGINEER
CHRISTCHURCH

PAGE S 18.
SECT
FILE
DATE

Ground Floor Actions XEQ SHEAR

col/wall No \ load case	D e=0 5	D e=emax 5	0·76D e=0	0·76D e=emax	1·0S emin 1	1·0S emax 3	0·57S emin	0·57S emax
1	48	209	36	159	137	212	78	121
2	48	209	36	159	137	212	78	121
3	387	350	294	266	613	538	349	307
4	66	467	50	355	274	424	156	242
5	861	893	654	679	1219	1324	695	755
7	494	488	375	371	725	729	413	416
9	491	469	373	356	741	703	422	404

Σ maxima = 1942

Ground Floor, YEQ, Shear

	D e=emax 5	0·61 D	1·0S emin 1	1·0S emax 3	0·57S emin	0·57S emax
1	662	404	1000	688	570	392
2	662	404	1000	688	570	392
3	144	88	93	26	53	15
4	1460	891	1300	1925	741	1097
5	113	69	143	49	82	28
7	83	51	17	13	10	7
9	119	73	66	37	38	2

Σ maxima = 570 + 570 + 1097 = 2237

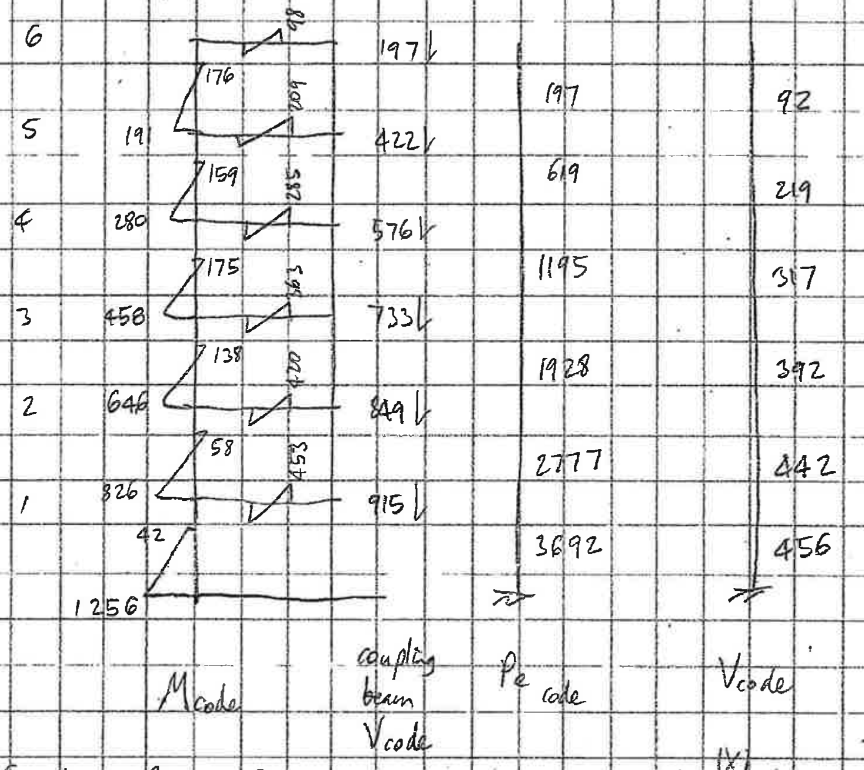
ie reduced static loads govern for each case.

total shear resisted = 0·57 × 3300
= 1881 kN ∠ OK

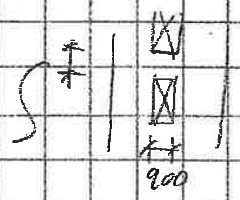
Hence design loads for walls 1 & 2

2 loads on p. S27 $\times 0.57 \times 0.80$

$= 0.456 \times$



Coupling Beam Design



levels 2 to 6 depth $= 1.24^m$
level 1 depth $= 1.70^m$

diag reinf. for $v_i = 0.1 \times \frac{900}{1240} \sqrt{25} = 0.36$ ((0.7)

at roof level $v_i = \frac{197{,}000}{0.85 \times 400 \times 0.8 \times 1240} = 0.58$

$\therefore$ all need diagonal reinforcement

2503
rec'd 1/9/86.

CHRISTCHURCH CITY COUNCIL

CITY WORKS & PLANNING DEPARTMENT

27 Aug 1986.

Alan M Reay Consulting Eng
P.O Box 25-028
Christchurch

Dear Sir,

Re Your application Number 1747 to erect office building
at 249 Madras St is held up pending receipt of:

rec'd on day or after letter sent

Please provide the calculations to support the design. We also require a foundation report and a Specification which describes the required quality standards for materials and workmanship. Please note that CCC Bylaw 105 requires in Cl 28.1. that "All drawings computations and other data submitted shall be signed by the architect, engineer or designer responsible for their production and shall clearly indentify him and his firm or organisation" There is no indication of on the plans that they have been checked and approved for issue and construction.

Please attend to the following matters:—

/ Sh 9 — No subgrade information and the 125 slab is both unreinforced and unjointed.

/ Sh 14 Stirrups for Cols 4, 20, 10 & 16.

/ Sh 15 Incomplete notes. Ref Line ①– Hi-Bond mesh reinforced encasting does not provide restraint to Hi-Bond for f.r.r purposes. also floor connection to shear wall system. and general connection between floor slab and walls.

/ S16 Shear Core floor slab & Stair landing details are missing.

S17 Thioflex 600 & PEF backing strip has not f.r.r.

/ S19 Not to microfilmable standards.

/ S23 Size of fixing A and we note that there are no notes.

/ S25 Reinforcing of spandrels and fixing details

✓ yes S26 Is there one. planter boxes & precast panel ✓

✓ S28 How is the web welded Smf... both sides

✓ S29 Detail 7 & 8 – 1 x 12d H.D bolt. No 2-M120

✓ S30 All weld plate details and Detail 2 stringer to weld plate weld size & type. also baluster fixings (see sheet S39.)

S32 Handrails & weld plate type 6 details.

Yours faithfully,

Gl Tapper

for CITY ENGINEER