A Submission to the Canterbury Earthquakes Royal Commission

Roles and Responsibilities

by John Scarry, BE(Hons), ME

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A. Personal Details

1. My name is John Michael Scarry. I am a structural engineer of Auckland, and currently work in sole practice.

2. I have a Bachelor of Engineering (First Class Honours) degree from the University of Auckland, specialising in structural engineering, and a Master of Engineering degree from the University of Auckland, specialising in structural analysis, structural dynamics and earthquake engineering.

3. I am the author of ‘An Open Letter to IPENZ on the Parlous State of the Structural Engineering Profession and the Construction Industry in New Zealand,’ 2002 (Appendix A of my submission on GEN.CERC.0003). This led, along with the revelations by the O’Sullivan brothers regarding leaky buildings and rotten timber, to the redrafting of the Building Act, leading to the Building Act 2004.

4. Referees

The following people form the core of leading engineers who have supported my efforts to reform the structural engineering profession and the construction industry for many years, along with John Henry, who I came to know two years ago:

(a) Associate Professor Charles Clifton, PhD, FIPENZ. For many years, Senior Structural Engineer at the Heavy Engineering Research Association. Chairman of the Steel Structures standard (NZS 3404) committee, and member of other standards and DBH committees.

(b) Dr Barry Davidson, PhD, FIPENZ. For many years, Senior Lecturer in the Civil Engineering Department of the University of Auckland. Managing Director of Compusoft Ltd, the leading structural analysis firm in the country.

(c) Colin Nicholas, FIPENZ. For many years, a successful consulting engineer, now ‘Designer in Residence’ in the Department of Civil and Environmental Engineering at the University of Auckland.

(d) Carl O’Grady, Chartered Engineer (UK), FIStructE. A pioneer of plastic design methods in structural steel, and the use of precast concrete in multi-storey construction.

(e) John Henry, CPEng, structural engineer of Christchurch.

It should be noted, however, that the following are my words, and mine alone.
B. Introduction

5. This submission is in response to the Discussion Paper: Roles and Responsibilities (GEN.CERC.0005).

6. It should be read in conjunction with my submission on Discussion Paper: Training and Education of Engineers and Organisation of the Engineering Profession (GEN.CERC.0003). To avoid repeating the lengthy appendices in that submission, this submission makes references to various appendices in my earlier submission. To avoid confusion, the appendices attached to this submission are numbered numerically, not alphabetically as before.

7. In response to Section 1.1 of this Discussion Paper, ‘Purpose,’ I state that the current regulatory framework is not only fundamentally flawed, it is fatally flawed, and oversees a building industry in crisis.

8. This crisis was first exposed by me in 2002 with my ‘An Open Letter to IPENZ on the Parlous State of the Structural Engineering Profession and the Construction Industry in New Zealand,’ 2002 (Appendix A of my submission on GEN.CERC.0003). This led, along with the revelations by the O’Sullivan brothers regarding leaky buildings and rotten timber, to the redrafting of the Building Act, leading to the Building Act 2004. Unfortunately, this is a fatally flawed document, because it was drafted by people with absolutely no understanding of the building industry who deliberately ignored every submission and recommendation I gave as to its flaws.

9. Unfortunately, the decade since my Open Letter has been wasted, and the shockers keep getting designed and built. This not only confirms my warnings regarding the profession and the industry beyond all reasonable doubt, the abject failure of ‘the regulatory framework’ to deal with this crisis for a decade proves beyond all reasonable doubt that the regulatory framework too is in absolute crisis.

10. “The tree is known by its fruit,” and the ‘fruit’ of this regulatory framework consists not only of all of the shockers exposed in my Open Letter and since (recounted at length in my submission on GEN.CERC.0003) and below, but all of the fatuous nonsense that has been presented as ‘reform’ since the Open Letter.

11. Radical, root and branch reform is needed immediately. There is no more time to waste, because the status quo is not static. The status quo represents a continuous decline in skill levels, competence, productivity and training that must soon lead to a complete collapse of the industry, because there simply will not be enough sensible, competent people left to turn the situation around, even with all the will in the world.

12. My use of the term ‘radical’ is somewhat misleading due to the unprecedented crisis we are facing. The ‘radical’ reforms that are required are intended to get the professions, trades and industry back to the sort of basic competence and soundness before the rot set in during the 1980’s.

13. The regulatory framework is, on the one hand, absolutely essential, and on the other, completely irrelevant. If the fundamentals of knowledge and practice and attitude in the professions of structural engineering, architecture, fire engineering and building services, and the trades of the construction industry were sound, and if these professions and trades were free to perform to their best without outside interference and restraint,
we would have no need for any building regulation whatsoever, other than some guidance as to the levels of design loading thought prudent.
C. Introduction to the Building Regulatory Framework

14. The Building Act 2004
The Building Act 2004 is fundamentally flawed. It is to a large extent a ‘nothingness,’ drafted by persons at the Ministry of Economic Development (MED) who knew nothing about the construction industry.

15. The Building Act 1991 was redrafted as a result of revelations about two crises:
- The revelations of the O’Sullivan brothers regarding ‘leaky buildings’ and rotten timber, and
- My Open Letter regarding the parlous state of the structural engineering profession and the entire building and construction industry.

16. Intended to address these crises, the re-draft partly acknowledged the ‘leaky’ crisis, having a focus on ‘housing,’ and almost completely ignored the ‘structural’ crisis. The creation of the essential technically competent body needed to sort these crises out was not done. Instead, a bureaucracy called the Department of Building and Housing (DBH) was formed, in much the same way as any other Government department dealing, say, with ‘Women’s Affairs’ would have been created, almost completely staffed by career bureaucrats, many ex-Treasury, and almost completely devoid of technical capability, especially at the Chief Executive and Deputy Chief Executive level.

17. My substantial submissions to the MED before and during drafting of the Bill that became the Building Act 2004 were ignored. (Refer Appendix 1 – some components omitted for space).

18. In my submissions to the Government Administration committee, I did try to get the Building Levy raised to at least 1% of the value of building work, at a time when building activity was sufficient to generate close to $80 million a year. This was to be funneled back into the industry by being used for direct training, scholarships to encourage ‘the best and brightest’ into university and trade training, and for direct testing and research. Unfortunately, I believe most of the Building Levy is consumed by the bureaucracy of the DBH.

19. My other extensive written and verbal submissions to the Government Administration committee that was tasked with reviewing this legislation were all ignored, as were my subsequent appeals to Parliament (My submission on GEN.CERC.0003 – Appendix F)

Since 1991, it has been stated ad nauseum that NZ has a performance based building code, instead of the previous prescriptive regime. This was repeated to this Royal Commission on 14 March 2012, I believe, during the discussions on New Technologies, and Professional & Regulatory Implementation. Many of these statements were made by ‘bureaucrats’ who have never used a structural code or standard in their life. I class these statements as ‘stuff and nonsense.’

21. What the Building Act 1991 did attempt to do was introduce a consistent nationwide building control system, to replace the previous By-Law based system by which each City, County or District Council set its own building regulations.

22. However, despite each City, County or District Council being able to set their own building regulations, as far as structural engineering was concerned they were all ‘much
of a muchness,’ and the latest loading, steel, concrete or timber standard was used in each area. The maintenance of such a system, without the loss of the City Engineers and Architects that went with it, may have stopped the ‘leaky building/rotten timber’ crisis before it even started.

23. With the exception of sanitary sewerage and fire engineering, in the ‘bad old prescriptive days,’ we had fully performance based building controls, especially as far as structures were concerned. Provided you could convince the local authority that what you wanted to do would work, you could do anything you wanted.

24. To disprove the claim that engineers were “imprisoned within a prescriptive regime,” I need only provide one example. It is a big example. The Stage 2 expansion of the New Zealand Steel Mill at Glenbrook in south Auckland consisted of the massive Hot and Cold Mills. At this time, the NZ design standard for steel structures was NZS 3404:1977. This incorporated AS1250 as the basic code, with some add-ons for seismic design. But most of the structural design for the Hot and Cold Mill buildings was done in Japan by Nikken Sekkei, using the Japanese AIJ Design Standard for Steel Structures 1979, a beautiful little code. This was allowed because Ken Patterson-Kane of Gillman Partners wrote a report to the local authority showing that the use of this standard and NZS 4203:1976 (subsequently 1984) would satisfy the local authority requirements for building safety.

25. Similarly, many other large industrial projects were designed throughout the country using a specially prepared document called ‘Seismic Design of Petrochemical Plants,’ a document I very much doubt was actually adopted by the local authorities within their By-Laws.

26. The concept of, and initial attempts at, seismic base isolation actually date back to the last quarter of the 19th Century. However, it is perhaps fair to say that the first base isolated building of the modern era was the William Clayton Building in Wellington. Designed by the Ministry of Works, it was a Government building I believe, and as such, would not have required a building permit. But I am sure that any private individual or company could have got such a building built at that time.

27. The New Zealand Building Code is Schedule 1 of the Building Regulations 1992. Far from being a ‘performance based code,’ I consider it nothing more than a ‘statement of good intent.’ I have extracted Clause B1 relating to ‘Structure’ (Appendix 2). Where are the quantifiable performance criteria against which a standard or design can be tested?

28. Far from being ‘performance based documents,’ loadings and design standards such as AS/NZS 1170, NZS 3404 and NZS 3101 are used in exactly the same prescriptive manner, day in and day out, for the overwhelming majority of the new building designs in NZ, as their pre-1992 predecessors were.

29. As a result of the ‘leaky building’ disaster, Acceptable Solution E2/AS1 for External Moisture was written. I dare anyone to try to get a Building Consent for a timber framed house without conforming completely and utterly with this document. The reality, as opposed to the rhetoric, is that this is every bit as prescriptive as the requirements for sanitary sewerage and fire engineering that applied in ‘the bad old days.’
30. **Compliance with the Building Code**
    
    As I expand on this in the rest of this submission – you can have ‘Compliance Documents,’ ‘Verification Methods,’ ‘Acceptable Solutions,’ ‘Determinations’ and ‘Alternative Solutions’ till the cows come home, but without sound engineering, architecture and trade skills being applied, they count for nothing.

31. All of the shockers described in my Open Letter, and exposed since then, have ‘complied’ with the ‘Compliance Documents,’ because all of the internal reviews, external peer reviews, producer statements and building consents said so. Except that they didn’t in reality, because the underlying sound engineering and/or architecture and/or trade skills were missing.
D. Efficacy of Building Regulatory Framework

32. The Discussion Paper states in Section 3.1.1 that the New Zealand Construction Industry Council (NZCIC) recommends the development of a ‘national policy statement,’ leading to a ‘regulatory hierarchy’ of:
   - Policy Statement on Building and Construction
   - Building Act 2004 & Regulations
   - Building Code
   - National Standards
   - Guidance Documents.

33. Blether, stuff and nonsense. Where is the ‘sound engineering’ and the ‘proper trade skills?’

34. I know of no loadings standard in the world that has to state that “gravity acts down.” Does that mean that engineers are uncertain as to which direction gravity acts? Of course not!

35. But I am aware of a certain reinforced concrete parking structure in England. The two way flat plate structure was designed using a computer program that I think automated one of the standard, non-finite element slab analysis and design procedures. Except that the engineer got his sign convention wrong, such that he had gravity acting upwards. Needless to say, the heavy reinforcing around the columns that should have been on the top was on the bottom, and the heavy reinforcing at midspan that should have been on the bottom was on the top. The structure got built like that, then started to develop large cracks!

36. In the late 1970’s and 1980’s, the steel structures standard was NZS 3404:1977. In reality, this was the Australian standard AS1250, with some seismic additions really written to cover multi-storey moment frame construction that was not occurring because of the boilermakers and the BNZ building in Wellington. A copy of AS1250:1981 is included in Appendix 3.

37. During the hearings on the CTV Building, ‘torsion’ came up a lot, in the form of twisting in plan under seismic loading, because the centre of mass of buildings is often offset from the centre of stiffness. Torsion on shear cores was brilliantly illustrated by John Henry and his ‘blue models.’ A steel beam that is subject to vertical and/or horizontal loads that are offset from the shear centre of the beam will have to resist the resulting torsion as well as the overall bending moments and shear forces. Significant stresses due to torsion can result, especially in open sections.

38. If you read AS1250, you will see absolutely no mention of ‘torsion’ (that is, primary equilibrium torsion), although there is mention of ‘flexural torsional buckling’ and ‘torsional restraint,’ which are different. Does that mean an engineer should then have ignored the stresses and deflections due to torsion? Of course not. Does that mean that an engineer could not have designed for torsion? Of course not.

39. By referring to a suitable text book, such as the excellent ‘Steel Designers Handbook’ by Gorenc and Tinyou (extracts in Appendix 4), an engineer could read up on the theory of torsion and its practical application, calculate the resulting longitudinal and shear stresses due to torsion, depending on whether the beam was open or closed, combine
them with overall bending and shear stresses, and then use AS1250 to check that these stresses were not excessive.

40. Where does this fit into:
   - Policy Statement on Building and Construction
   - Building Act 2004 & Regulations
   - Building Code
   - National Standards
   - Guidance Documents?

   It doesn’t, and it doesn’t have to. It is the engineering, and the subsequent skilled construction that are essential – without them, everything else counts for nothing.

41. I dealt with IPENZ in my previous submission, and will not waste any more time on it.
E. Background to the Regulatory Environment

42. Here I shall briefly repeat some of what was stated in my earlier submission, and expand upon it.

43. Historically, NZ structural engineering has suffered from two serious problems. The public’s ignorance of what engineers are, and the importance of what they do (typical of most English speaking countries), and the fact that whereas virtually any part of NZ could be hit by a devastating earthquake at any time, the period between large earthquakes in built up areas is so long that flawed practices are not exposed and proper seismic resistant practices cannot be developed or enforced.

44. Following decades of continuous post-war growth, a sharp decline in the economy in 1967 severely affected the construction industry. I am told that the resulting demand for redundancy payments for carpenters led to changes in the employment of carpenters which had severe ramifications in the following decades.

45. A cycle of booms and busts continued during the 1970’s and early 1980’s, with two oil shocks having a major effect, followed by the Think Big projects. Contractors cannot help themselves from wanting booms, but a boom and bust cycle is the worst thing that can affect a construction industry, and no developed country has been so affected as NZ. One minute there is no work for skilled people, so they leave, next minute, anyone with a pulse is working in the industry. It is a recipe for disaster.

46. Treasury has long been completely staffed by laissez faire economic loons whose economic theories are detached from all reality, and don’t even work in theory. Unfortunately, they have been given free rein by successive Labour and National led Governments, to the point where Treasury has caused at the very least one third of a trillion dollars of damage to the New Zealand economy. Treasury has devastated the productive and skill base of this country, none more so than in the massive and vitally important building and construction industry.

47. For the following, I am indebted to Tom McRae, a 91 year old true gentleman who is near death, but I hope he hangs on long enough to read and hear this.

48. In May 1983, Treasury began a ‘Review of Planning and Building Controls,’ investigating an industry about which it knew nothing, knows nothing, and cares to know nothing. (Refer Appendix 5).

49. This review included such gems as:
   - “If a person wished to design an unusual residence, e.g. with bedrooms connected by one metre diameter tubes ......., should he be prevented from doing so?”
   - “If a building owner desired to alter the interior of his residence by removing bearing partitions to the danger of himself and his family in the next storm, should he be prevented from doing so by the by-laws which say he cannot without a permit? At present he is prevented from doing anything to a partition, whether bearing or not, unless he has a permit to do so. His castle is not his own.”

50. They are lunatics. But not only were the lunatics let loose, they have been running the asylum ever since.
51. A former President of IPENZ has brilliantly stated that in the 1980’s “There were two queen bees in The Beehive – Treasury and the Ministry of Works and Development. One of them was going to be stung to death, and it wasn’t going to be Treasury.”

52. The MOWD was tasked through legislation with providing independent economic advice to the Government, and with care of the state of the entire public infrastructure of NZ, including local body infrastructure. As a result, it was a rival to the laissez faire loons at Treasury, and their plans.

53. Between 1984 and 1993, Treasury, aided and abetted by the likes of Douglas, Prebble, Palmer, Moore, Caygill, Richardson and Birch, visited the following devastation on the New Zealand construction industry:
   - The MOWD was dismantled, with the ‘intellectual property’ flogged off to Malaysians,
   - The input of the MOWD into maintaining high standards of practice was lost,
   - The role of the MOWD in properly training vast numbers of engineers, technicians and especially draftsmen, a role that underpinned the private sector, was lost,
   - The similar role that Government departments such as NZED, NZ Railways and the like played was lost,
   - Highly competent design departments (which never designed a ‘leaky building’) at bodies like the Ministry of Education were destroyed,
   - The often highly competent and very efficient Engineering and Architecture departments of territorial authorities were destroyed. This has severely eroded the ability of the territorial authorities to adequately perform their duties as Building Consent Authorities, and
   - In an act of unabashed madness, Bill Birch destroyed the centuries old apprenticeship scheme, that had served this country, and especially the building industry, so well.

54. The insane financial deregulation that accompanied this economic terrorism led to a building boom up to 1987, during which time a vast array of ‘rubbish’ buildings were designed and constructed, followed by the crash and the deliberately orchestrated severe recession of Ruth Richardson.

55. And if all that was not enough, due to ‘penny pinching’ on the part of clients, especially developers, the critical role of Clerk of Works was destroyed. Clerks of Works were highly skilled in inspecting construction and administering the contract on site. On any reasonably sized job, a Clerk of Works would be on site full time, representing the interests of the owner, but also the integrity of the building, and all subsequent users of the building.

56. [Deleted]

57. Other ‘reforms’ of this nature saw universities becoming partial businesses, which had to tout for fee paying students (who often ‘hint’ that they have paid their money, and want the degree they have paid for), and the technical institutes, which had long trained apprentices of world class, morphing into pseudo-universities. These ‘institutes of technology’ are now highly variable in quality. Some have reached the point where
universities are about to no longer recognise their qualification when considering applicants for direct entry into the last two years of the BE degree.

58. NZ used to send apprentices overseas for international competitions. Refer to Appendix 6. This article from the NZ Herald of 1 October 2004 shows how things have declined.

59. Germany has the best trade training in the world, and believe it or not, many of these German qualifications are not recognised by the likes of AUT because they are “not degrees.” Germany has very high wages, and maintains a massive trade surplus with the rest of the world year after year, especially in manufactured goods.

60. Not only is Treasury the ‘queen bee,’ it is a parasitic wasp, having laid its eggs throughout the upper levels of the civil service.

61. The Ministry of Economic Development (MED) is nothing of the sort, and is a second arm of Treasury. The MED made a complete mess of the Building Act 2004, and ignored all of my suggestions and submissions. The Department of Building and Housing (DBH) that was created as a result of the Building Act 2004 is almost completely devoid of technical capability, and has failed to effectively recognise, let alone effectively deal with, the crisis in structural engineering and construction in NZ.

62. In an absolutely nonsensical move, the DBH has recently been absorbed in the new ‘super-ministry’ called the ‘Ministry of Business, Innovation and Employment’ (MBIE), along with the Ministry of Economic Development, Ministry of Science and Innovation and the Department of Labour.

63. I predicted the ‘power’ within this new ‘super-ministry’ would be the MED, and who was then appointed as the acting Chief Executive of MBIE? The Chief Executive of the MED, surprise, surprise. There is no way the gross technical deficiencies of the DBH can be made good by absorbing it into an organisation run by the MED.

64. I shall address the technical deficiencies within the DBH in more detail later, but to give you an idea of the paucity of the critically essential technical competence that now prevails in the ‘technical departments’ of the civil service, consider the New Zealand Transport Agency (NZTA).

65. The Chief Executive of the NZTA was the Chief Executive of the MED when they drafted what became the Building Act 2004. He is ex-Treasury, surprise, surprise.

66. In early 2011, ‘out of the blue’ an engineer I know sent me a copy of a draft design guide called ‘Steel/Concrete Composite Bridge Design Guide, NZTA Research Report TAR 09/04.’ Steel/concrete composite bridges have a main deck structure consisting of steel beams supporting a concrete slab, with the two interconnected in such a way that the steel beams and concrete slab form composite beams stronger and stiffer than the steel beams alone. The Heavy Engineering Research Association (HERA) was involved in developing this guide for the NZTA.

67. I know enough about bridges to know they are far more difficult than they appear, and try to stay clear of them, in the same way that a bridge engineer would stay clear of buildings. I did not want to get involved, but felt obliged to have a look. I had serious concerns about what I read, especially the section on the two way slab behaviour of the deck slab. I didn’t have the time, but felt obliged to pass on my concerns to HERA, and
wrote the 7 pages of comments attached in Appendix 7. The focus of my comments related to the two way slab provisions I covered in my Items 4 to 6. This dealt with ‘the basics’ of slab behaviour, behaviour so basic even I could understand it.

68. As a result of these comments, I was asked by HERA to become involved in completing the design guide. I made an initial, then an updated, offer of services in February 2011, and waited for a response. And waited. In May 2011, I chased things up. My initial contact at HERA had left without telling me, so I contacted one of the more senior staff.

69. I was told that NZTA had only received comments on the points I made from me, and no one else. Apparently, NZTA had no one who could assess my comments as to their technical merit. NZTA was waiting for other people to raise the same issues that I had before engaging me to work on the development of the guide!

70. NZTA has a multi-million dollar, tax payer funded budget to administer a multi-billion dollar, tax payer funded budget to maintain and expand the national roading network, including thousands of bridges. Despite the enormous funds provided by the NZ taxpayer, and the obvious technical competence required to perform its function, the NZTA was incapable of reviewing clearly stated comments dealing with fundamental aspects of bridge deck design. Instead, the NZTA was going to determine its actions based on a popularity contest. The argument ad populum, only worse.

71. Clearly, the NZTA considers that it needs not engineers, but experts in popularity. May I suggest the NZTA goes on a recruitment drive amongst American teenage high school prom queens. I withdrew my offer of service because the situation was untenable.

72. As if the devastation visited upon the structural engineering profession and the construction industry by the likes of Treasury and the MED over decades was not enough, further devastation has been visited by the likes of the Commerce Commission, and various ‘commercial’ treaties, such as GATS, and various ‘free trade’ deals.

73. Up until the mid-1980’s, in general a minimum scale of fees applied to structural engineering design fees, based on a percentage of the total building cost. At this time, fees were about 3% of the total building cost, for what were usually relatively simple, repetitive commercial buildings. Given the amount of work required to earn these fees, the gross and net income per employee per hour was by a clear margin the least of all recognised professions, despite the enormous responsibility that went with every job.

74. The scale of fees was determined by the profession on the basis of the actual hours it took to do real jobs properly, and the very modest hourly rates assigned to the engineers and draftsmen employed by the engineering firms.

75. But such ‘collusion’ was anathema to the likes of the Commerce Commission, who did not, indeed could not, appreciate that:

- Design and documentation (geotechnical, structural, architectural and building services) is not the totality of what is being provided to the client, and proper design and documentation is critical if the total cost is to be minimised,
- Unclear, uncoordinated and mistake riddled designs and drawings increase the cost and endanger the structure out of all proportion to the cost of producing sound, competent designs and drawings,
- The enormity of the amount of work structural engineers have to do if each job is to be done correctly, and the fact that they are invariably doing one off,
unique designs, not ‘buying things in bulk and adding on a large mark up,’
which constitutes so much of ‘commercial activity’ in NZ nowadays, and

- The potential horrific consequences if the engineer gets things wrong, and
- The critical ‘life safety’ aspects involved, particularly when dealing with the
enormous forces that earthquakes impart on buildings, and
- The fact that ‘the end users’ constitute vast numbers of people over many,
many decades.

76. Having completely no understanding of these critical issues, or the fact that the entire
infrastructure of any modern economy is completely and utterly dependent on the work
of structural engineers, the Commerce Commission outlawed the minimum scale of fees.
The result of this, and ‘throat cutting’ throughout the profession, has led to the collapse
in fees where in many cases they are bid as lump sums little more than 1% of the total
cost of construction, and sometimes down to as low as 0.6% by some of the least
technically competent consultants. Add in the fact that many of these lump sum bid jobs
‘turn to custard,’ with massive amounts of design work that cannot be claimed for, and
you have a recipe for absolute disaster – the devastation of the profession, and the
collapse of major buildings. That is precisely the situation we have now. And where
was the Commerce Commission to be seen when the Waitakere Trusts Stadium and the
Vector Arena were collapsing, and Stadium Southland collapsed? Oh, “it’s got nothing
to do with them.” I disagree. The Commerce Commission, Treasury, the MED, all of
the negligent politicians and the like are every bit as responsible as the engineers
concerned.

77. Not content with fees and ‘bureaucracy,’ and the ‘bloat’ of properly training engineers
and tradesmen, the ‘theoretical economists’ of Treasury, MED and the Commerce
Commission then turned their attention to ‘standards.’ Obsessed with the idea of
removing ‘barriers to trade,’ they have pushed for joint Australian – New Zealand
standards. I shall cover this in more detail later, but will note the following. Unlike
most other areas of engineering, such as mechanical and electrical, very significant
differences have developed over the last 50 years between the Australian and New
Zealand construction industries, especially in reinforced concrete. Their codes are based
on the British BS 8110, ours are based on the American ACI 318. Cast in-situ
construction predominates in Australia, precast (unfortunately) predominates in NZ.
And, of course, seismic design is much more important in NZ. Yet the forced joint
Australia – New Zealand concrete reinforcing steel standard has been a disaster for this
country, as I explain later.

78. My reason for hurrying to get the Second Version of my Open Letter to the Government
and Opposition in December 2002 was the threat that the much talked about expansion
of the General Agreement on Trades in Services (GATS) posed. The thought of ‘free
trade’ in engineering design with the likes of Communist China and India, countries with
extremely low costs and appalling records with regard to building performance during
earthquakes, was too frightening for even me to contemplate. Even in Taiwan, modern
concrete buildings, supposedly designed to good American practice (at 0.25% design
fees), have collapsed in recent earthquakes because empty kerosene cans had been used
as illegal ‘fillers’ in the middle of reinforced concrete columns, to save a few dollars on
the concrete.

79. I was soon told by the bureaucrats that with the full support of the large NZ consulting
firms, the National Government had signed engineering services up to ‘free trade’ under
GATS in 1994. We now have NZ consulting firms outsourcing design work to
Indonesia and India, and such centres of structural engineering excellence as Myanmar and India are advertising design services in NZ at rates as low as $US15 per hour for engineers.

80. But wait, there’s more. Consulting engineering firms used to have to be, by law, partnerships with unlimited liability. In the 1980’s, these firms were allowed to become limited liability companies. That, in and of itself, was not necessarily a problem, although clearly from what I have exposed the rot had already set in. But since then, many of these NZ engineering firms, especially the large, multi-disciplinary firms, have been sold off to foreign owners, and in some cases, partially floated on the stock exchange.

81. This is completely unacceptable. Consulting structural engineering is not a business, it is a profession, or at least is supposed to be. The duty of care an engineer should exercise goes far beyond his/her fee paying client. It is a duty of care that covers anyone who could in anyway ever be affected by the performance of the engineer’s design and supervision, and in most cases, extends over several lifetimes.

82. Whilst in the first instance the foreign owners of these once NZ firms may be engineering firms in their own right, their owners, or their owners owners are not engineering firms, but ‘investment companies’ or ‘private equity’ concerns, who in reality view these engineering firms as nothing more than profit generation centres. This is completely unacceptable.

83. It will be claimed that this foreign ownership allows the NZ subsidiary to benefit from contact with ‘the best’ international expertise, and NZ offices have access to international projects they would otherwise not have. Both of these claims are nonsense. The decline in standards that has occurred for decades in NZ has affected these large multi-discipline firms as much as any, if not more so. And prior to this nonsense, and GATS, many NZ firms did a lot of work overseas.

84. Also, this slide towards ‘commercialism,’ as opposed to professionalism, has led to the dominance of the ‘management and marketing’ culture, and the emergence of the cult of ‘leadership,’ as I explained in my submission on the Discussion Paper: Training and education of engineers and organisation of the engineering profession (GEN.CERC.0003). These ‘cults’ have seriously eroded the standards of engineering design and supervision. An earthquake doesn’t give a fig for the management structure of the company that ‘designs’ a building.

85. I cannot delve too deeply into this subject here, except to take a quote from ‘The Winning Performance,’ by Clifford and Cavanagh:

> As Reynold M. Sachs, who was present during the turnaround of Digital Switch and made some $50 million in a three year period, said .... “Ironically, individuals whose primary or only goal is financial gain end up failing to make money that they dreamed about ...... because it’s short sighted and gives you incentives to make the wrong decisions.”

To which I would add “unless you are a bankster.” If we are to have a sound construction industry in NZ, capable of designing efficient buildings that do not fall down under their own selfweight and are not munted in even moderate earthquakes, we have to get back to where structural engineering is a profession, free to do the engineering to the highest standard, paid the professional fees necessary to do every job on its merits.
F. The Standing of Structural Engineers, Fees and the Effect of Defective Design Documentation

86. Consistent with most English speaking countries, particularly the most ‘British,’ until the Canterbury earthquakes occurred, the majority of New Zealanders did not have the slightest understanding of what professional engineers do, or the importance of their work, and this was most true for structural engineers. Everyone knew that architects made buildings stand up, and ‘engineers’ shoveled coal into the boiler of a steam locomotive.

87. The ‘standing’ of engineers, especially structural engineers, is critically important, not from a point of view of pride or arrogance, but in order for them to be able to do their work effectively.

88. I would first like to point out the importance of engineering as a whole to a modern economy. Unlike NZ, where the most popular university courses are ‘Psych 1, Sociology 1 and Anthropology 1,’ the most popular university course in Germany, across all departments, is mechanical engineering. Germany is a manufacturing and export powerhouse, despite having very high real costs and wages. Even in seismic zones outside Germany, if German structural engineers are involved in designing and supervising construction to American based codes, those buildings invariably perform well. Proper trade training in Germany is mandatory. If a company is found not to be pulling its weight, for example, not training enough apprentices given the size of the company, it is fined heavily, and the money is used to subsidise the other firms’ training schemes.

89. Germany’s manufacturing export supremacy is based not only on the large companies like Daimler Benz and the giant chemical concerns, but the many mid- to small-sized mittelstand companies that are export-oriented and contribute to Germany being the world's second largest exporter. They focus on innovative and high value manufactured products and occupy worldwide niche market leadership positions in numerous segments. They are typically privately owned, often by the same family over several generations, and based in small rural communities.

90. Invariably, these giant companies, and the mittelstand, are run by engineers with doctorates in their field of engineering. But there is a very important twist. Obviously talented students, before they went off to university to study for their Bachelor’s degree, they invariably served an apprenticeship to fully understand the entire process of design and manufacture. These senior directors do not hesitate to roll their sleeves up and show young apprentices how to do things properly, should they see something untoward going on.

91. Contrast this with what goes on in NZ consulting engineering firms. For many years, with one minor exception, I never saw a senior manager, most of whom trained as structural engineers, ever browse through the drawings lying around in the structural section, just to see the true quality of the work being produced.

92. In the late 1960’s, there was a series of border clashes between the Soviet Union and Communist China. The unthinkable happened. Soviet troops froze to death in the snow, because they did not know how to sleep in the open with just a greatcoat for protection. That this should happen to Soviet or Russian troops was a national disgrace. Soviet Generals, all veterans of the Second World War, left their heated opulent offices and
holiday homes, went to the front, and taught the troops how to survive in the ice and snow. Despite the appalling examples in my Open Letter and since, which normally involved equally rudimentary failings, I know of no example where a senior engineering manager in a large firm has ‘rolled up his sleeves,’ rounded up the graduates and intermediate engineers, and said “Look, this is how you must do it.” That would be genuine leadership, and it’s just not there.

93. It is not just that structural engineering ranks so far below the likes of medicine and law as to recognition, influence, control of its own destiny, and income. Even within structural engineering firms, the actual design and drafting is treated largely as something that is ‘trivial,’ that anyone can do, and it must be churned out so as to meet the deadlines and the inadequate fees. The structural engineers are considered a cost, not an asset. “Why are you taking so long?” “You’re costing too much money?” “Let the contractor sort it out?”

94. Who’s saying this? The structural engineers who got into ‘management and marketing.’ But who’s going to do the work properly, if this is the fate of senior engineers who stay being designers? Contrast this with surgeons, who are in the operating theatre until they retire, or in this Royal Commission, the senior counsel, even the Queen’s Counsels, who operate ‘at the coal face’ until they retire, and are paid for their expertise and experience, not their ‘management roles.’

95. In the next section, I deal with the unsustainable constraints that are imposed on structural engineers by the other ‘professional and trade’ parties involved in construction industry, but wish here to touch on the perceptions of clients.

96. The effective charge out rates of dentists, doctors and lawyers are, in almost all instances, considerably in excess of structural engineers. Dentists are usually able to do quite a lot of good work in half an hour, so the total bill is relatively small, although still painful for most people. Obviously, people will pay a lot for specialist medical advice, either directly, or through the public health system, because their life is on the line, the specialist or surgeon is doing much of the total work, and ‘the system’ recognises the overall skill levels and years of training of the doctors concerned. Minor legal matters such as conveyancy can usually be done quickly, and the cost is perceived as a small percentage of the total sums involved. For more major cases, people are forced, and are prepared, to pay a lot to ‘fight the fight,’ especially if they can tip a 50:50 outcome more to their side.

97. But with structural engineering, the attitude is invariably “Buildings are supposed to stand up, so why should I pay for it?” combined with a misunderstanding that contractors know what to build anyway. Until things go wrong, of course.

98. Engineers cannot ‘bury their mistakes,’ and they can’t pass off shortcomings as “Well, that’s litigation.” Also, we are involved in design, which must by its very nature be an iterative process. We are also involved in what are often highly constrained, one-off designs, often on difficult ground. Other civil engineers, such as harbour, dam, geotechnical and roading engineers are often able to design a large value of work in a relatively small amount of time because they don’t have to deal with all of the detail and minutiae that structural engineers do. This is not to say the risk involved with their work is not great, because it is, but they do not usually have to design the myriad of small but important details structural engineers have to.
99. Many houses nowadays require at least some important input by structural engineers. Some houses are major structural engineering projects, largely built off the engineer’s drawings and ‘coloured in’ off the architect’s. The ‘average’ house in Auckland has a design life of 50 years, and is sold once every 12 years on average. Let us assume a realistic life of 72 years, which means it will be sold 6 times. Let us go back to the 1990’s, when one ‘average type’ house I encountered cost about $200,000 to build excluding the land. At 3.5% per sale, the real estate agents will take a total of 6 times 0.035 times 200,000 = $42,000, or 21% of the total value of the house, in fees. For this house, I offered on behalf of my employer a fee of $1,500 for us to design and draft retaining walls under the house, some beams, and some bracing elements, a fee of 0.75%, or one 28th of what the real estate agents would take out of the house over its design life. I was told by the ‘architectural designer’ that that was too much, and $500 was the ‘going rate’ for the engineering on such a house. We would have done a proper job for the $1,500, but is it any wonder that 75-90% of ‘leaky homes’ are found to have serious structural defects unrelated to water damage upon investigation. This cannot be allowed to go on.

100. Another project involved the design of three complex luxury townhouses on one site. The middle one was two storeyed, the ends ones three storeyed, and the front of each sloped, so that there was no repetition between townhouses or even between each floor. These ended up costing over $4 million to build. With no input from me, a design fee of $20,000 was tendered, and this was not the lowest bid – that was $15,000. At this time, an accurate rule of thumb was that the total cost for overheads, meetings, design time, drafting, checking and profit to produce a drawing was $2,500 per drawing on average. In other words, the winning tender for structural engineering work assumed only 8 drawings would be required, for three completely different townhouses. You couldn’t draw the floor plans for that. Plus the original programme allowed only 3 weeks for the design! Three engineers and three draftsmen busted their guts to get it done in about eight weeks, with a total of 55 drawings, and $120,000 on the ledger, or a loss of $100,000. That is completely unsustainable – is it any wonder the profession is in crisis? Despite the extremely complex nature of the townhouses, in a seismic zone, the total reinforcing in the masonry walls was little more than code minimum. The architect, who was specifying genuine gold fittings in some of the bathrooms, complained that we had used too much reinforcing.

101. In another luxury house project in Auckland, a demanding $2 million concrete and masonry house was built, had to be built, off the engineer’s drawings, and was ‘coloured in’ off the architect’s. The engineering fees were a little over 1% at $24,000. These had been beaten down from $30,000 offered by a competitor. The architect, who would have been getting at least 4 to 6 times the engineer’s fees, if not more, said engineers “Had to get realistic on house fees.” I have seen a completely collapsed timber framed modern townhouse in Christchurch, which would have killed anyone who had been inside on 22 February 2011. That is the true realism, and it cannot be faced properly at the typical fees being paid to structural engineers for such work.

102. Moving to the commercial sector which is more in keeping with this Royal Commission’s Terms of Reference, in the early 1980’s structural fees tended to be around 3% of the total cost of a new building, and except for the largest shareholders in some of the largest firms, engineers were hardly ‘rolling in it.’ Despite more complex codes and ever more demanding architectural concepts, fees have typically collapsed to be in the 1% to 1.5% range. Around 2000, several companies in Auckland were competing in the commercial market for mid-size office buildings and larger warehouses
at around 1% fees. One of them, the worst as regards technical competence, then cut fees down to 0.6%. This company has designed some absolute shockers. This cannot go on, but it does, and no one is stopping it.

103. I have mentioned modern Taiwanese buildings, designed by engineers for 0.25%, which collapsed in earthquakes, at least because the contractors cheated on the construction.

104. On April 23, 1988 the rooftop parking deck of a brand new supermarket collapsed at Station Square in Burnaby, British Columbia, Canada. Five minutes after the opening ceremony, four bays of the roof structure came plummeting to the ground. Despite there being an estimated 600 customers and 370 employees in the building when signs of failure started to occur, no one was killed, but 21 people were injured. Refer Appendix 8.

105. A Commission of Inquiry found that the collapse occurred due to multiple severe design deficiencies. The building had been designed for a structural engineering fee of 0.25%, and that was not the lowest fee tendered for the job! I have not been able to locate a copy of the findings of this Inquiry, but did read in an article at the time that the Commission recommended that structural fees should be a minimum of 6% of the total building cost.

106. Fees in NZ are at present typically around one quarter of this minimum, and I am sure will not increase markedly for the rebuild of Christchurch, despite all the additional work that will be required to ensure good diaphragm design alone.

107. When I have discussed fees in a general sense with quantity surveyors, architects and engineers I know personally, a common response is that the structure is only about one third of the total cost of the building, and why should an engineer get paid for things like architectural finishes (but they never mention that the architect’s much higher percentage fee includes foundations and structure they have nothing to do with). It doesn’t matter whether one considers the minimum necessary structural fees as say 10% of the structure cost or 3% of the total, the present fee levels are at a level that guarantees a profession and industry in crisis. It is also my experience that the more ‘expensive’ the architecture, either because of expensive finishes or extreme geometric complexity, the more complex the engineering involved, and this workload can easily increase geometrically if the work is going to be done properly. Fewer and fewer buildings have simple repetitive structures.

108. The need for greater fees is not so that all engineers can go out and buy the latest Porsche. It is so that enough good people can be attracted to the profession, properly trained, have a satisfying career, there is ‘fat’ able to be reinvested to improve productivity, and so that the quality of the work can be improved, to lower the overall cost of construction.

109. As the likes of Shewart, Deming and Juran showed to the Japanese, true productivity and lower costs can only come about through quality. What is lost on the likes of the Commerce Commission and Treasury is that if you grind structural engineering down to its present parlous state, instead of building costs dropping as engineer’s fees collapse, the total costs go through the roof.

110. Attached in Appendix 9 is a report prepared by the Queensland division of Engineers Australia in October 2005. ‘Getting It Right The First Time’ looks at the total cost of
buildings as the quality of project design information decreased from the mid- to late’
1980’s. Not surprisingly, this decline started as soon as professional fees declined, and
why did professional fees decline? Because the Association of Consulting Engineers
Australia’s (ACEA) scale of minimum recommended fees was discontinued under
pressure from the competition authorities.

111. As an aside, which shows the complete failure of such ‘competition authorities’ on both
sides of the Tasman to deal with the real ‘rip offs’ in the economy, in the Australian food
retail sector a duopoly has developed to the point where consumer prices are high, but
just about every Australian food producer has gone or is about to go bankrupt, because
the duopoly has driven what they will pay the producers into the ground.

112. Consistent with the significant warnings I made in my Open Letter (warnings which one
prominent engineer described as “an irrational attack on the ‘free’ market”), the
substance of this report can be found on the pages labeled ‘6’ and ‘7.’

113. Since the 1980’s, in Australia engineering design work has been largely “awarded on
price rather than value and capability.” [For NZ, I would add “awarded on price, and
being prepared to do whatever the contractor wants.”]

114. Extensive research for this report showed:
   • Design efficiency has a non-linear inverse relationship with project design fees.
   • Project costs due to design inefficiency increase sharply when design fees are
     reduced below the cost of doing the work properly.
   • The concept of reducing total project costs by increasing expenditure on the design
     process has been well-documented through principles of value engineering and
     value management.
   • 60% to 90% of all variations are due to poor project design documentation.
   • Poor documentation contributes, on average but conservatively, an additional 10-
     15% to project costs.

115. In 2005, the total Queensland size of the construction industry was $A 20.7 billion. The
report estimated that the direct waste or loss due to the low standard of project
documentation (and certainly aided and abetted by declining trade skills) was between
$A 2 billion and $A 3 billion. The economic multiplier effect caused the total loss to the
Queensland economy to be even greater, at $A 5.8 billion to $A 8.7 billion.

116. The report found that standards continue to decline, and that:
   • Poor documentation leads to serious underperformance within the industry,
   • A significant waste of resources,
   • An inefficient, non-competitive industry,
   • Cost overruns, rework and extensions of time,
   • High stress levels, loss of morale and reduced personal output,
   • Adversarial behaviour and a lowering of professional reputations,
   • A potential decline in safety standards,
   • A decline in the viability and sustainability of the industry.

117. The report goes on to state that proper project documentation should be:
   • Fit for purpose,
   • Unambiguous and coherent,
   • Timely, accurate and complete,
Easily communicated and constructed, with the best possible economy and safety, and
Aligned with the owner’s requirement.

118. The report writers could have been reading from my Open Letter.

119. To this, I would add that the corresponding decline in standards in NZ has not only led to enormous waste, without even considering the multi-billion dollar ‘leaky building’ crisis, but much worse. It has led to the construction of large numbers of buildings that will perform far below expectations under seismic loading, to say the least, and many large buildings that cannot even safely carry their modest design gravity loads, even their own selfweight. And no one is doing anything to try to address this crisis, apart from me and my close group of supporters.

120. Whereas other professions such as accountancy have promoted themselves into fields that they should perhaps keep out of, structural engineering has retreated, to the detriment of all.

121. The consequence of leaving dimensions off drawings, for fear of making a mistake or the architect changing something, being too scared to think about how the structure should best be constructed, for fear of ‘liability,’ submitting to the expectations of the lowest common denominator in the construction industry, retreating from on-site supervision, along with the decline in fees and genuine technical excellence within engineering firms have been to not only grind down the structural engineering profession, but the construction industry as a whole, and the wider economy. It cannot carry on like this. It has to stop.

122. It is not just in skill levels, fees and GATS that the interference of the likes of Treasury and the Commerce Commission has seriously eroded the productivity of the construction industry and the quality and safety of the resulting buildings. Appendix 10 contains a 2005 Australian Steel Institute discussion paper ‘Are You Getting The Bolts You Specified?’

123. The standard high strength structural bolt in Australia and New Zealand for decades was a domestically manufactured Grade 8.8 bolt. For a couple of dollars, you could buy an incredibly strong and ductile, well-made bolt, nut and hardened washer assembly, the components of which had been forged, heated, quenched and tempered to have an ultimate tensile strength of 800 MPa. Most of these bolts were made by Ajax GKN, and Australian and New Zealand engineers often had to worry about how the bolts were installed by contractors, but they never had to worry about the bolts themselves. Occasionally, similar grade bolts could be sourced from the US or Japan, but again, high strength bolt quality was never an issue. These bolts were not only strong, an M20 being able to carry almost 10 tonnes in shear, they were ductile, and could be safely fully tensioned and still be expected to resist anything an earthquake could throw at them.

124. But a flood of ‘cheap’ imports drove Ajax out of manufacturing in Australasia around 2001. Unfortunately, many if not most of these Chinese manufactured ‘high strength bolts’ were not worthy of the title ‘structural bolt,’ let alone ‘high strength.’ In 2001, Chinese bolts were fracturing after assembly in a large portal frame building in Victoria. In late 2004, an aircraft hangar collapsed, and the investigation showed that not only were the bolts not up to standard, “certification with any validity was hard to obtain.”
125. In 1999, the US government enacted Public Law 101-592 (1990), also known as the Fastener Quality Act (FQA). This legislation forced US importers to take responsibility for the product they were supplying into the US market. The law was introduced after defective and counterfeit fasteners caused the death of nearly 400 US citizens over 15 years.

126. High strength structural bolts should be manufactured by the following process. High quality bar stock is cut off and cold formed in machines that are fitted with expensive, precision made hardened dies. The resulting shaped bolt must then be carefully heat treated to a precise uniform temperature, quenched in an oil bath that is maintained at a precise temperature, then tempered by controlled heating and air cooling.

127. In many of the Chinese factories, and this excludes the out-and-out counterfeiters, what happened was:

- Defective bar stock was used,
- The dies in the forming machines were never of the correct shape and hardness, or were well past their service life,
- Instead of being fed through the initial heating furnace in a well spread out single layer, a mass of bolts piled high was being fed through, so many of the bolts never received the necessary heating,
- Frequent power cuts severely affected this controlled heating,
- Instead of being dropped in small batches into the quenching oil baths so that the temperature of the bath could be kept down to its required level, large batches would be dropped in to already overheated oil, so that the required quenching was never achieved.

128. A seriously defective critical structural product was the result, and yet it was able to virtually eliminate Australasian manufacturers who had produced a world class product that could be relied upon without an engineer giving it a second thought.

129. In NZ, some small producers like Boltmasters continue to make bolts, but cannot make any money out of them.

130. Appendix 11 shows some rectangular hollow section products. Rectangular hollow sections are like a rectangular or square pipe. A flat sheet of steel is carefully formed by rolling, and then closed as the abutting edges are electro-resistance welded to form a through-thickness weld. Finally, the outside face of the weld is ground smooth, to produce an efficient, high quality structural product. (Sometimes, a round pipe is formed and welded first, and then the pipe is forced through rollers to produce the final shape). Or at least that is what is supposed to happen, and for many decades, European, North American, Japanese, Korean and Australasian manufacturers could do just that. Examples of proper hollow sections are shown in the first page of Appendix 11. The remaining pages show the defective Chinese product, a critical structural product, being shipped into California.

131. In a North and South article (April 2009 from memory) dealing largely with the ‘rotten timber’ crisis, one person interviewed described how Chinese shower units consisting of untreated plate glass were being imported into this country. These are completely and utterly illegal, because when the glass breaks, it will slice any poor unfortunate in the way to shreds. Why are these allowed into the country, when they must be intended to be installed and used? It falls on a building inspector to notice that it is plate glass, although I wouldn’t be surprised if it’s marked as ‘safety glass, complying with ......”
Another job for the building inspectors – don full protective gear, then bash each shower cubicle with a hammer. What has this country come to?

132. If that wasn’t bad enough, as a result of a complete balls up in the drafting of the combined Australia/New Zealand standard for concrete reinforcing bars, a whole raft of reinforcing we did not want in this country because it is unsuitable for real world seismic applications has flooded the market. In this case, most of it is not ‘counterfeit’ or ‘defective’ as such, it is just of a type that is not suited to seismic resistant design. But more on that later.

133. If all that wasn’t enough, cheap Chinese imports of stainless steel fasteners such as bolts for exterior handrails in coastal zones have driven out all competition, and are the only such fasteners available now. Except most of them are grossly sub-standard, and this will lead to a multi-million dollar ‘corroding stainless fastener’ scandal in a few years’ time. This is not the world I was born into, and I do not like it. It is not the world I trained in, and I do not like it.

134. This ‘corruption’ of building materials is putting us back into the darkest days of the mid-19th Century, when structural engineers could not trust the critical building products like cast iron sections and wire cables. Numerous major collapses occurred as a result. Even where trustworthy alternative products are still available, the structural profession does not have the time, the fees, and most importantly, the skilled staff, to run around trying to verify that not only is the ‘paper work’ for the structural materials in order, but the materials themselves are up to standard. This is especially so on small to medium size jobs.

135. Not only do we have the farce that far from the Commerce Commission, Treasury and the MED driving down construction costs they have massively increased them, these ‘competition authorities’ have not addressed the fact that despite having an absolute abundance of raw materials and energy, NZ has about the highest priced building materials in the developed world, because of monopoly or near monopoly control of the supply chain. Not to mention a $22 billion ‘leaky building, rotten timber’ crisis the materials suppliers have largely escaped having to pay for.
G. The Building Act 2004 and Its Review

136. To recap on my previous submission and the above, the Building Act 2004 is not fit for purpose, particularly with regards to the structural engineering crisis and the skills crisis in the industry. It should be thrown out, and new legislation written.

137. What has been repeated in Section 3.1.2 of the Discussion Paper constitutes yet further evidence of the complete malaise and deceit that entraps the construction industry. Most of what is mentioned relates only to house construction, but since it is introduced, I shall address it.

138. The Building Act 2004, (including the Building Code) was reviewed in 2009. According to the Cabinet Minute (CAB Min (10) 27/10 refers), the review aimed to:
   - Clarify and simplify building regulatory requirements and require a more targeted, risk-based approach to their administration by building consent authorities; and
   - Clarify the responsibilities of building producers to residential consumers, and better equip residential consumers to transact with confidence for building work.

This applies only to residential work, and completely and utterly ignores the Waitakere Trusts Stadium, the Vector Arena, and all of the other ‘post Open Letter’ shockers that were occurring.

139. The review found that the building regulatory system is not broken, but that it is costly and inefficient. The review did note that changes made by the Building Act 2004 had contributed much-needed improvements to the quality of building work. However, several areas for further improvement were identified, including:
   - problems ensuring responsibility sits in the right place;
   - weaknesses in consumer protection; and
   - undue reliance on building consent authorities.

The Building Act 2004 only improved the worst aspects of the ‘leaky building’ crisis, and did not address the structural issues, from the smallest houses up to the largest buildings. ‘Costly and inefficient’ is the same dangerous rhetoric that was put forward in the 1980’s and 1990’s when the construction industry was devastated deliberately by successive Governments and Treasury. The bullet points apply only to household units, but the 2011/2012 changes to the Building Act do not even address these problems, as I describe below.

140. The rest of the statements in Section 3.1.2 are straight from the Government and DBH/MBIE, and they are absolute nonsense. With regard to the much heralded ‘improvements for home owners’ embedded in the Building Amendment Bill (No 3) (passed as the Building Amendment Act 2012) and the Building Amendment Bill (No 4):
   - The defined roles and responsibilities are no different than they have been under common law for centuries,
   - Nearly all of the shockers described in my Open Letter and since had written contracts ‘for Africa,’ and that didn’t stop them being shockers,
   - If things ‘turn to custard,’ the owner will still have to spend many tens of thousands if not hundreds of thousands of dollars in the courts, arbitration or the WHRS in seeking damages, money most owners simply do not have and can never fully recover as costs, and
   - Claims for defects under the Building Act are required to commence within 10 years of the act or omission. There is absolutely no requirement under these
amendments for ‘the Contractor’ to still exist after 10 years, or have any money, or have insurance against such claims, or have insurance that survives for 10 years.

141. But it gets worse. By what must be a deliberately deceptive and misleading ploy, a CCC is no longer a CCC. In many, many cases involving ‘leaks,’ rot and structural defects in houses and apartments, the only thing that has saved the owners from complete and utter financial ruin has been the fact that the ‘local Council’ has been ‘the last man standing.’ By getting rid of the ‘CCC,’ that is, the ‘Code Compliance Certificate,’ and replacing it with a ‘CCC,’ that is, a ‘Consent Completion Certificate,’ the Government has manoeuvred the ‘local Councils’ largely out of being ‘last man standing.’ In future, affected owners will face financial ruin, period.

142. This ‘review and reform of the Building Act’ is incompetence and deceit rolled into one, with the crazy mindset of ‘red tape’ removal thrown in, the type of mindset which largely led to all of the defective house construction in the first place. The lunatics are running the asylum, and no one charged with a fiduciary duty to protect the interests of the people of New Zealand is doing anything to stop it. Extracts from my submission to the Local Government and Environment Committee on the Building Amendment Bill (No 4) are attached in Appendix 12.
H. Roles and Responsibilities – Part 1 – Recap of Shockers

143. I shall deal with the parties mentioned in Section 4 of the Discussion Paper under my Section J. In this Section H, I wish to recap on a ‘few’ shockers. In Section I, I deal with crucial parties involved in day to day building projects, and address roles and responsibilities not covered in this Discussion Paper, in fact, I believe, not covered by the Royal Commission at all up to this point, although I stand to be corrected.

144. To recap on a ‘few’ shockers:

145. The ‘leaky building, rotten timber’ crisis, that has/will not only affects tens of thousands of homes and apartments but also numerous school buildings, with a total cost of at least $11 billion if not $22 billion dollars.

146. The fact that 75-90% of these ‘leakers’ are found, upon investigation, to have serious structural deficiencies, unrelated to water damage.

147. All of the shockers contained in my Open Letter of 2002 (my submission on GEN.CERC.0003, Appendix A).

148. An incomplete summary of shockers since the passing of the Building Act 2004:
   • The Waitakere Trusts Stadium (2004), which was massively overstressed at the point where the main roof trusses landed on their support columns. The roof was failing during erection because it could not carry even its own self-weight through these connections.
   • The original design of horrendously ‘cut off’ shell beams, only 225mm high, designed in the office of a ‘pre-eminent designer of precast concrete,’ which was condemned as incorrect and unsafe by NZ’s two top concrete engineers, and the two co-inventors of shell beams, yet was approved by two Fellows of IPENZ, and lauded by a senior member of the NZS 3101:2006 committee.
   • The Vector Arena, which came within a hair’s breadth of collapse due to at least two major deficiencies. But for a miracle, it almost certainly would have failed in service, and could have come down on 12,000 people.
   • The large shopping centre concrete floor slab diaphragm, which was designed as if ‘simply supported’ by walls at both ends, yet which in fact had to cantilever off walls at only one end.
   • Another massive floor in the same complex where, after having been shown how to analyse an L shaped floor diaphragm by the reviewing engineer, the ‘designer’ proceeded to ignore the massive forces at the re-entrant corner because he considered them “errant forces.”
   • The large shopping complex in another city that apparently required rebuilding of concrete foundations, concrete beams and steel superstructure during construction, because of design defects.
   • The large educational building that had been designed and peer reviewed, yet had no viable means of resisting lateral seismic loads in the foundation system, had a transfer diaphragm at ground level that had not been analysed or designed, just assumed to be ‘infinitely strong,’ had a soft storey, had not had the main seismic analysis that was done by others checked, and that had grossly deficient stairs, including a feature stair that would certainly have collapsed as initially designed, and probably would have collapsed as redesigned by a Chartered Professional Engineer, until I pointed this out.
- The major bridge whose seismic analysis and design did not make sense to the owner’s reviewer, until it was pointed out that the analysis defied all common sense, and instead of the bridge deck acting as a rigid diaphragm in the plane of the deck, it was wobbling round ‘like a blob of jelly.’ Even when fully explained to them, the designer and his superiors could not understand this absolutely fundamental deficiency.

- The large concrete and steel bearing plates for large ground anchors that formed a very basic structural system, yet two engineers could not analyse, design or detail them properly.

- The Australian supermarket building designed by a NZ firm that has failed due to eccentric steel cleat joint failure, a design detail that only I ever seemed to think about, until I got Charles Clifton interested.

- A similar roof failure in Whangarei.

- The Pepperwood Mews apartment complex in Waitakere City. The thirty two unit complex constructed from reinforced concrete is not only a leaking, sodden mess, it is so structurally unsound, it is a ‘wrecking ball’ job. It was designed in 2005.

- The 4m high concrete retaining wall for a motorway that had an opening joint that would have ripped open as ‘detailed.’ Leaving aside the lack of any proper joint design, the ‘not to scale’ drawing hid the fact that the reinforcing would have been poking out of the concrete at the front and bottom of the wall/foundation joint if the hooks had been properly developed from the critical sections.

- The multi-storey apartment building in Auckland, designed circa 2005, where the following defect was only found because the building as a whole was a ‘leaker,’ including leaks through cracks in an external structural shear wall. This structural wall was designed and constructed as an assemblage of precast panels, acting compositely with heavily reinforced and confined end columns. Heavy horizontal reinforcing projected, with hooks on the end, from the ends of the precast panels, and had to be embedded in the end columns, something that could not actually be done because of the very heavy column reinforcing. How did the contractor fit the panels in place and complete the wall? He cut most of the projecting starter bars off, of course. This may have ended up as a ‘site shocker,’ but that was the inevitable consequence of the initial design deficiencies.

149. Failed or illegal ‘buildings of the rebuild,’ covered in Appendices B, R & S of my submission on GEN.CERC.0003.

150. The failed dance floor from the ‘rebuild.’ (Appendix 13).

151. The CTV Building, the Grand Chancellor and the Forsyth Barr Building.

152. All of the other multi-storey Christchurch buildings designed and built in the 1980’s and later that have been demolished since the earthquakes. These displayed all sorts of potentially catastrophic failure modes which were not supposed to have occurred. If the earthquake of 22 February 2011 had been less intense, but had gone on for a minute or more, many of these buildings would have suffered significant collapse (these buildings have not been covered adequately by the Royal Commission – it is a miracle that some of them stayed standing).

153. The 28 storey Majestic Centre in Wellington, dating from 1991 (Appendix 0 of my submission on GEN.CERC.0003). Essentially an earthquake prone building.
154. Several apartment buildings in Christchurch, including the one that was on Park Terrace, that have had to be demolished (Appendix 14). These apartment buildings, as is often the case throughout NZ, seemed to lack structure in one direction, because of the desire to maximize ‘views.’

155. These shockers are not just the product of a structural engineering profession and a contracting industry in crisis, they are products of the entire building ‘system,’ and in these sections, I wish to address the parties to that system, other than the Government, Parliament, and the Building Consent Authorities.

156. Given the above shockers, perhaps ‘Role Playing and Irresponsibility’ would be a more appropriate title for these sections.
I. Roles and Responsibilities – Part 2 – Parties Involved in Actual Projects

157. In this section, I wish to cover the roles and responsibilities of those parties that are involved in most construction projects, and directly affect the ability of the structural engineer to properly perform his/her duties, especially with regard to seismic resistance.

158. These parties are the clients, the Local Authority planners, the architects, the project managers, the quantity surveyors, the contractors and the sub-contractors.

159. Before doing that, I wish to use a boxing analogy to address the problems structural engineers face in trying to just do their job properly. Except for perhaps one question asked by Mr Rennie and one by Mr Mills QC, the Royal Commission appears to have assumed that structural engineers alone are responsible for the layout of buildings, especially their seismic resistant systems.

160. By 1987, ‘Iron’ Mike Tyson was the heavyweight boxing champion of the world. Nearly every victory of his career had been by knock out, most of them in the first round. He was so intimidating that his opponents were invariably defeated before they stepped into the ring. Until, on 11 February 1990, the 42 to 1 underdog, James ‘Buster’ Douglas stepped into the ring. Douglas wasn’t intimidated, and he had the jab to win. He stopped Tyson’s attacks in their tracks, and knocked the previously undefeated Tyson out in the 10th round to claim the title.

161. Like Tyson, large earthquakes have historically demolished everything they are of a mind to. Today, a competent structural engineer, free to apply the best American, Japanese, NZ and other seismic resistant practices can defeat the earthquake. But like Douglas, the engineer has to be free to fight his best fight. In New Zealand today, before he could ‘step into the ring,’ the structural engineer would have his feet bound together and his hands tied behind his back by the client, the project manager, the planners, the architect, the quantity surveyor, the contractor and the sub-contractors, and still be expected to win. The situation is untenable, even for the few ‘Buster Douglas’ types we have left.

162. Clients

In general discussion about the problems with the NZ construction industry, ‘developers’ are often held up to be chief culprits. I have generally avoided such statements, for no other reason than the worst shockers I have described have had no ‘developer’ involvement. They were mainly designed for ‘blue chip’ institutional clients by ‘blue chip’ firms.

163. However, it is true that ‘developers’ have played a part in the ‘low cost fee, do anything the developer or contractor wants’ driven, corrosive decline in engineering standards. After things ‘turn to custard,’ developers often feign ignorance, and claim to have thought they were employing ‘the best,’ when they knew full well they were employing ‘the convenient,’ because, after all, one Building Consent is the same as any other, until of course the structure is hit by a snow load or an earthquake.

164. The ‘reforms’ heralded in by the Building Act 1991 put a lot of the responsibility for decisions as to the ‘quality’ of a building onto the owner. But invariably, what do the owners know? How can a typical citizen getting a house built be expected to make informed decisions regarding the structure, particularly when they don’t understand why they should have to pay more than a pittance for the structural engineering? How can a
school board made up of parents be expected to be an informed purchaser of architectural or engineering services? If presented with two conflicting sets of structural engineering advice, how can they be expected to follow the more correct one? Many dozens if not hundreds of school buildings are ‘leakers’ and slated to replacement.

165. The naive observer would assume that a very large company whose business is that of developing, owning and leasing commercial property would dismiss clearly incompetent engineers, and employ competent ones, albeit at fees that have been driven down to an unsustainable level. Well, the naive observer would be wrong. I know of one such company that continues to employ one consulting engineering firm that produces shocker after shocker. The client knows this, but appears to be following a strategy that this ‘cheap’ firm will produce ‘cheap’ designs, and another engineer who is engaged to review these designs and find the deficiencies will save the client from disaster. This ‘checking’ engineer reasonably expected to be employed to actually do some competent original design, but this has not occurred, and he has since stopped carrying out these reviews.

166. **Planners**
Sound seismic resistant design is not something that can be ‘painted on’ after the planners and architects have fulfilled their wish lists. In the Auckland CBD in particular, although it is not completely their fault, planners have allowed the built environment to be devastated with shoddy office and apartment buildings, and overall, have allowed mindless urban sprawl and infill ‘densification,’ yet impose numerous constraints that drastically inhibit sound seismic design and construction.

167. I do not know if there is a ‘planners general theory of building heights,’ but I suspect that the building heights allowed in District Plans for the CBD of NZ cities are based on the same ‘storey height’ assumptions that apply in non-seismic Sydney, London and Paris. In these cities, it is structurally acceptable to design ‘flat plate’ slab and column reinforced concrete structures to resist gravity and lateral loads, or at worst with some assistance from some very minor walls. Such things are not possible in seismic zones, and reinforced concrete frame structures especially require sufficient storey heights to accommodate the deep beams required, the services that must run under the beams, and steps in the floor slab that are often required.

168. Clients do not want to pay for any more storey height than they have to, because of the cost of it, but engineers often need more than they are allowed, if they are to provide a truly safe seismic resistant structure. Building heights need to be defined in terms of ‘number of reasonable storeys,’ and not height in metres.

169. This is especially true for the capacity design of seismic resistant structures, and especially those with podium structures and transfer diaphragms. Everything cannot be properly sized and thought about at the preliminary design phase, especially for geometrically complex buildings, and the proper final design must not be arbitrarily constrained. The CTV Building was very simple in form, and built on a flat site with no basement, and look what happened to that, and all the similar 1980’s office buildings that have since had to be demolished.

170. I am aware of extremely complex transfer diaphragms in Auckland at least, full of steps and riddled with penetrations, which were far from well designed. They would have needed weeks, even months of work from an experienced engineer to design them properly (I am serious and can prove it). If during such a final design, the engineer
determined that the total thickness of the transfer diaphragm floor structure had to increase by 200-300mm, and hence the entire building height increased, that must be allowed. Otherwise, any pretense to proper seismic design is just that, pretense.

171. “Some of my best friends are planners, but...” Well, in actual fact, I am acquainted with quite a few, and have got on well with them. In my own experience, leaving aside height restrictions, they have been very reasonable to deal with, as far as their District Plans allow. However, there is a serious failure in planning at the highest level as it affects seismic resistant design, and Christchurch is no exception. Please refer to Appendix 15. This contains a submission Charles Clifton and I made on the Christchurch City Council’s (CCC) Draft Plan for the rebuild of the CBD.

172. You should be aware by now of Charles Clifton, and the excellent performance of well-designed multi-storey braced steel buildings on 22 February 2011. I drafted this submission alone because Charles was too busy, but he happily co-signed it because he agreed with it. I may be an engineer of no importance, but he isn’t.

173. This CCC Draft Plan was developed by the CCC planners without, as far as we can tell, any input from a structural engineer, let alone any sound input. This for a CBD that had just been devastated in a series of earthquakes, and a command from the Mayor that the rebuilt city must be “the safest city in the world, from a seismic perspective.”

174. From what I can see, the much heralded Christchurch Central Development Unit (CCDU) ‘100 Day’ revised central city plan is largely the original CCC plan, with some large ‘public’ buildings thrown in to sweeten the pot.

175. Our submission’s Appendix A includes artists’ impressions of the proposed rebuilt CBD streets, and these reappear in the CCDU ‘100 Day’ plan. These impressions show terraced three to five storey buildings, side by side, generally with a ground floor containing a cafe or restaurant or shop, and with apartments above. Perfectly fine for Sydney or London or Paris; not so good for Christchurch.

176. Clearly, these buildings must be ‘open’ front and back, and have full depth, full height concrete or masonry fire walls to both sides. Therefore, with a few exceptions where an owner and architect can accept cross walls or cross bracing, these buildings must have moment resistant frames in the transverse direction (parallel with the footpath) to resist seismic loading. Such frames are inherently flexible as structures go. To avoid the potentially catastrophic ‘pounding’ of adjacent structures, it is an absolute legal requirement that no part of a structure sway outside the confines of the legal boundary under an earthquake, if there is an adjacent building. ‘Honest calculations’ will show that each fire wall should be set back from the legal boundary something like 100-150mm for the buildings shown, meaning that there should be 200-300mm ‘alleyways’ between all of the buildings, but they are not shown.

177. These ‘alleyways’ can indeed be provided, but how are they to be kept clean of rubbish and vermin? I suspect that in many cases, these mandatory seismic separations will not be provided.

178. Also, the need for double fire walls, not to mention the offset piles and hence foundation beams required to support them, not to mention the multiple stairs and lift required for the apartments, is going to result in massive costs and reduced leasable floor areas. We made the innovative suggestion that if all the separate legal titles on a block were
amalgamated, a single ‘structure’ (with continuous floors) could be built that would only need single fire walls between each ‘strata’ title, with a single row of piles under each wall. The individual ‘buildings’ could be made to look different, and one of the suitable ‘buildings’ could have transverse bracing or shear walls to properly resist seismic loads in that direction.

179. Of course, our submission has been ignored by two ministers, one mayor, one chief executive of CERA, one chief executive of the CCDU, one city council, one city’s planners, several ‘business leaders leading the rebuild’ and everyone else. But I have come to expect this. When questioned about these matters by the Christchurch media, one of the senior city planners said “The public can rest assured that all of these building will be built to the highest structural standards.” Yeah, right. Even if they are, at what unnecessary cost?

180. Also, the soils of the CBD are not too bad, but cry out for vibrocompaction, which must be done over wide areas, and cannot be confined to one property, especially if neighbouring buildings have already been built. Even piled buildings can be expected to differentially settle 100mm at least if this compaction is not done. It may be that the layout of the remaining buildings and buried services will preclude this, but it should have been considered from day one, and it wasn’t, and my suggestions regarding this have been ignored. I was even informed that vibrocompaction “may not be suitable for the soils of the CBD,” when it was developed for those types of soils in the first place, and has been performed very successfully at the Waikato North Head mine site and at Tiwai Point for the main chimney foundation in the past.

181. The Mayor is talking about “the next 300 years.” Piles are a form of ground pollution. With all of these piles under the terraced buildings of the CBD re-build, what happens in 100-150 years’ time when one of these buildings has to be replaced. The steel in the piles may have corroded significantly, and the piles could be undersize or in the wrong place. What then?

182. Another serious problem that planning rules can cause with respect to the seismic resistant design of multi-storey buildings relates to ‘brownie points’ for certain features desired by planners. I was once required to work on the preliminary design of a building, after numerous other engineers had had some input. For NZ, this was a tall building, and relied on central lift core walls and perimeter walls for lateral load resistance. To get to the desired building height under the planning rules, the architect had provided a ‘garden’ at a ‘podium’ above the street. In order to provide light at this ‘garden,’ some of the perimeter walls were stopped off for one storey at this level! This created a soft storey, and I seemed to be the only engineer to have raised the critical point. Fortunately, the project never went much further.

183. Architects
Architects, the bane of the seismic engineer. Some of my best friends are architects, really, but I must make the following observations. As time has gone on, more and more evidence has accrued as to the need for a symmetric layout of stiff lateral load resisting elements on all four sides of a building to resist seismic loading, and the need for positive load paths, especially proper connection of floors to lateral load resisting elements. The constant drive for ‘more economic’ buildings implies the need for rational repetitive building elements, so that each element can be ‘optimised,’ and then the cost savings of repetition can be achieved. This is particularly important for NZ,
where the Treasury induced devastation of the economy has meant we can no longer afford the ‘extravagance’ that true First World economic status may allow.

184. Attitudes are everything in these matters. I know architects who are reasonable and prepared to listen to a structural engineer, but the problem with them is they have usually laid out their building and shown it to the client before getting any input from the structural engineer. I also know of architects who treat engineers as something they try not to step in, although they still expect the engineer to take full responsibility for the resulting design.

185. One example of the nonsense engineers now have to put up with relates to the interaction between one architect and one good engineer in a multi-disciplinary firm. This senior architect had been working on modifications to an existing building, and had removed large parts of a long central wall that not only provided lateral load resistance but was also required for gravity support, in order to suit the desired architectural layout. This ‘wall removal’ was done without any consultation with or input from a structural engineer. This architect went along to a senior engineer at about 2pm on a Friday afternoon, saying that “the design is going out to the client at 5pm, can you do the engineering?” Quite rightly, the engineer told the architect to “go away.”

186. In other instances in the same firm, responsible architects were tragically let down by grossly incompetent structural engineering design.

187. Traditionally, NZ university trained architects were very well regarded overseas. One FNZIA who I had a very close, productive working relationship for many years, told me that when he went overseas to the UK in the late 1960’s, he could find employment because he knew how to detail a real building, unlike the typical Cambridge or Oxford architectural graduate. He had been taught at university how to detail a door, a window and a flashing, along with a rudimentary understanding of structure,* and during his work in NZ for some leading consultancies, he had always been required to detail all of the architectural elements properly.

188. *This is fortunate, because in one of the instances I came across after my Open Letter was written, this architect was finding gross mistakes in the bending moment diagrams and subsequent reinforcing arrangements being presented to him by a completely incompetent engineer! Again, I swear, I am not making this up.

189. Once a stand-alone school like the School of Engineering, the University of Auckland School of Architecture has been absorbed as a department into the ‘National Institute of Creative Arts and Industries,’ along with the likes of music, dance and fine arts. Last time I heard, this ‘institute’ was run by a musician. Now I love listening to music, but a musician or a dancer should not be in control of a school of architecture. Another senior architect I know says that architecture is not art, it is about building buildings.

190. Unfortunately, the ‘art’ governs, to the point where, despite our economy sliding down the rank of countries year by year, even minor jobs are becoming complex three dimensional ‘works of art’ best reserved for the likes of the Guggenheim Bilbao. Despite the ‘art,’ most of our large buildings, especially in Auckland, look shocking.

191. The sensible lay observer would assume that in response to the ‘leaky building’ crisis, there would be a massive increase in practical training at the schools/departments of architecture. This has not occurred, certainly at Auckland. I was reliably informed that
about 5 years after the leaky building crisis was exposed, the last vestiges of practical
training in the form of carpentry and woodwork were removed from the curriculum!
Last time I heard, the total time allocated in the 5 year degree to understanding the
structural aspects of the non-specific light timber framing code (used for nearly all
houses and many small commercial buildings as well) was four hours. Not four hours a
week or semester or even year. Four hours per degree.

192. I repeat – sound seismic resistant design is not something that can be ‘painted on’ to a
finished building. Often, without any engineering input, an architect will lay out a
building with no clearance to the boundaries, irrationally thin walls without essential
column and pilaster thickenings, a completely eccentric lateral load resisting system,
often complete with soft storeys, no allowance for temporary retaining, or piles that
could never be installed shown hard up against existing buildings. Shear walls are often
placed to the outside of exterior stair wells, or riddled with irrational openings in the
lower storeys, or all of the lift, stair and services floor penetrations are placed adjacent to
the walls such that proper connection of the diaphragms to the walls cannot be achieved.
This is all exacerbated by the relatively tall buildings now being put onto the relatively
small NZ commercial CBD lots. This can’t go on. All designs must have input from a
structural engineer at the very start of the butter paper concept stage.

193. Numerous earthquakes have shown that in addition to a symmetric layout of lateral load
resisting elements being required, with such elements on each exterior face, if non-
structural damage is to be minimised for non-base isolated buildings, then these lateral
load resisting elements must be stiff. In other words, they must be large frames, or
preferably shear walls or braced. The only way to get this symmetrical stiff arrangement
may be to have strict ‘symmetry’ criteria in the codes, and where these are violated, base
shears are doubled or trebled.

194. Project Management

Project management is an essential skill that competent architects, structural engineers
and Clerks of Works used to be able to display in abundance. Since around 1980, the
‘profession’ of stand-alone project management firms has not only developed in NZ but
taken a strangle hold on the construction industry, with much cost and no benefit to
show for it. The observant onlooker will note that this ‘rise of the project manager’ has
coincided with ‘the development of the crisis in design and trade skills.’

195. In the United States, large project management companies used to earn their fees by
replacing the ‘general contractor,’ and directly handling the complete coordination of all
of the sub-contractors. In NZ, project managers have come along to add their 3.5% to
4% fees to the project cost, without in most instances replacing the ‘general contractor.’
(Structural engineering fees are typically less than half that paid to the project
managers).

196. These project management companies are usually bereft of technical capability, and
must refer all technical questions back to the engineer or architect. They claim to be
able to ‘ask the hard questions’ but never do.

197. I don’t want to waste any more time than I have to on them, but will describe two
projects:

- On one very complex, luxury town house project, the first project manager
developed a design ‘program’ thus. He looked at the time from the present to
when the clients wanted their townhouses. He took off from this period the time
required for the clients to make up their mind as to the basic layout of the townhouses and the site. He then subtracted from the program the time he estimated was required for construction. That left three weeks, which he declared to be the time for design to start at the architects and finish at the engineers. This was just absolute nonsense, and the many dozens of detailed structural drawings required took months to prepare under extreme pressure, and massive fee overruns occurred that could not be recovered. ‘Headhunted’ overseas, this project manager was replaced, and after the design had been completed in complex reinforced masonry covering many dozens of pages, the replacement project manager asked glibly “Could it be redesigned in precast?” He didn’t have a clue, other than to think that precast was “cheaper and faster.”

- On another project, an existing building was being massively modified to carry loads it was never meant to carry. New piles had to be driven through holes cut in existing suspended ground floor slabs. When told that a 55 tonne tracked piling rig could not drive over the thin suspended floor slab and drive the piles off the slab, the project manager asked, in all seriousness, “But how do they drive down the road?”

198. One would have thought that finally, as a result of the Canterbury earthquakes, ‘the public’ and ‘government’ would finally recognise the importance of structural engineering. No. A local authority recently announced one of its major buildings was earthquake prone, and had to be seismically strengthened. So who was appointed as ‘lead consultant?’ A firm of structural engineers? No, a firm of project managers.

199. This is what one former President of IPENZ describes as ‘management without content.’ It is the antithesis of ‘the German success story.’

200. This ‘retreat’ by the structural engineering profession as a whole, handing the design phase over to the complete dominance of architects and now project managers, has not only damaged the structural engineering profession, but has severely compromised seismic design standards and overall technical standards throughout the industry, not to mention productivity.

201. **Quantity Surveyors**

Quantity surveyors have an extremely important role to play in the construction industry, scheduling quantities and estimating total costs. Unfortunately, some quantity surveyors have tried to expand into a role as ‘lead consultant’ on projects, a task they are not qualified to do, because they are not able to make the critical technical judgements that (competent) architects and engineers can.

202. Quantity surveyors were involved in nearly all of the shockers I have recounted, and if not the houses then certainly the leaky apartment buildings and leaky school buildings that must now be replaced. They have pushed certain construction practices as ‘the cheapest,’ including many that have come to grief in the Christchurch earthquakes, practices that I have been trying to get banned or modified for years.

203. Quantity surveyors have largely been unable to identify bad practices, have been unable to address the issue of monopolistic building material supply chains that make building materials in NZ horrifically expensive when compared with Australia and North America, and have not been able to address the low standards of productivity within the NZ construction industry, nor the waste due to design and documentation mistakes.
204. I know architects who have lost long term clients, for whom they have performed outstanding service over many years, because of unfounded throw away comments by quantity surveyors. We have to get back to the fundamental basics of sound practice within the construction industry, and that means that quantity surveyors should ‘survey quantities,’ and leave the wider issues to other professionals who, in turn, have to get back to the competent practice of fundamental basics.

205. **The Building Contractors**

   I know many fine people working in the contracting industry, from the smallest one man carpentry firms up to the largest contractors. On an individual basis, these people agree fully with my efforts for reform. In contrast, one large company that employs a number of Chartered Professional Engineers regularly has outside speakers along as part of the internal CPD process. It is perfectly acceptable for this firm to invite directors of companies that design the shockers I expose, but I am banned, because the company “cannot be seen to be agreeing with me,” even though, if I turned up and talked nonsense, the engineers would not hesitate to say so, and tell me where I could go.

206. Despite these fine individuals, the productivity of NZ building firms, especially on commercial projects, is very low by First World standards. All of the factors that have affected structural engineering have adversely affected these contracting firms, along with the destruction of the trade training schemes, and the damage has been done. Many top grade firms are let down because they simply cannot get the skilled staff, such as proper welders, that they need. And instead of having the best welders out of the Korean shipyards migrating here, we invariably have unskilled labourers coming in.

207. Leaving aside the long known about, but only recently ‘discovered in NZ’ dynamic fracturing of reinforcing, probably my biggest source of disappointment in my career has been that the opportunity to design a sound seismic resistant cast in-situ reinforced concrete building has been denied to me because of the devastated skill base of the NZ construction industry.

208. Whether seismic resistant or not, I could design a heavily reinforced, large concrete building in Australia, Canada, the US, Britain, France, Germany, Sweden, and most of the rest of the world, including Asia, and have no problem getting it built quickly and economically using modern formwork systems. But not in NZ.

209. Refer to page 3 of Appendix λ in my submission on GEN.CERC.0003. This shows in-situ slab formwork and extremely heavy beam and column reinforcing for a seismic resistant building under construction on the west coast of the US. Slab reinforcing is yet to be placed. The Americans can construct this, all in-situ, at a rate of four days per floor. The NZ construction industry struggles to throw up the most appallingly conceived and detailed precast concrete at a rate of one floor every seven days. In New York or Germany, if you can’t build a cast in-situ reinforced concrete high rise at a rate of one floor every two days, you can’t compete.

210. **Sub-Contractors**

   Again there are exceptions, but sub-contractors, including those doing work off site, suffer from poor and declining skills and productivity. You will have got the picture by now, but I shall refer you back to the unhooked spirals in Appendix V of my submission on GEN.CERC.0003. These cages were constructed off site by a ‘leading’ supplier of reinforcing, despite what was specifically and clearly shown on the drawings, and DBH Practice Advisory 8 that I had got the DBH to release several years previously.
211. Several years ago, I designed 36 small diameter piles for a new house project. All of the piles were meticulously and clearly detailed. Five or so cages would turn up at a time, but even if they were all supposed to be identical, they would not only contain basic mistakes in the cages, but the mistakes would be different for each pile! Common was the lack of hooks on the spirals, so the reinforcing supplier would send out his staff to bend the hooks, which they did well. Then, a day or two later, the next batch of pile reinforcing would arrive, complete with missing hooks and other defects, and on it would go.

212. Engineers are at best only paid for ‘observation,’ not ‘supervision,’ there is a lot of work, especially precast we simply cannot inspect, because even if we had the fees, we do not have the skilled staff, yet we can be held liable for any and all of this bad work we might miss, despite our best efforts at ‘observation.’

213. On 2 August 2012, I went down to Wellington to appear before the Local Government and Environment Committee regarding the Building Amendment Bill (No. 4). The person appearing before me was an acquaintance of mine, a senior engineer employed by one of the large building contracting firms. We had a long chat afterwards, and he mentioned the issue of Licensed Building Practitioners and off site work. I have not checked up on this aspect of it, but he said that whereas LBP’s were required to do, say, reinforced concrete work on site, they were not required to do the same work off site, in the form of precast.

214. Through his involvement in different industry bodies, he hosted some DBH officials on a site where precast reinforced concrete beams were being connected using in-situ concrete at the joints. He said to them “Let me get this right. You don’t have to be an LBP to make the formwork, place the reinforcing, and place and compact the concrete for the precast beams in the precast factory, but you do have to be an LBP to do the little bit of reinforced concrete in the joints?” “Yes, that’s right.”

215. **Fees and What Engineers Have to Do**
   Typical structural engineering fees for large seismic resistant, commercial type structures are now at about 1% to 1.5% of the total project cost, no matter how complex the structure. Some fees are down well below 1%. These are far, far lower than the typical architectural and project management fees, and often less than the fees charged by the quantity surveyors.

216. The architect has to do few if any supporting calculations, and although he/she may have to draw the same object several times in different views if not using 3D computer modeling, there is no such thing as ‘multiple load cases’ for each object. Similarly, quantity surveyors have to measure each object once, the contractor is required to build each element once, and the project manager only has to consider the construction of each element once.

217. Structural engineers are usually highly constrained geometrically by the planners and the architect, and often have to deal with less than ideal soil conditions for which foundations must be designed often before anything other than a preliminary design of the superstructure can be completed. Let us consider how many load cases the structural engineer must consider for strength, ignoring stability load cases, any relevant staged construction, and retained soil loads and ground water and snow loading, assuming that seismic response is fully elastic ($\mu=1$) or nominally ductile ($\mu=1.25$):
• 1.35G
• 1.2G + 1.5Q (Full)
• 1.2G + 1.5Q (Skip 1)
• 1.2G + 1.5Q (Skip 2)
• 1.2G + \psi_cQ + W_{ux}
• 1.2G + \psi_cQ - W_{ux}
• 1.2G + \psi_cQ + W_{uy}
• 1.2G + \psi_cQ - W_{uy}
• 1.2G + \psi_cQ + W_{ux}
• 0.9G + W_{ux}
• 0.9G - W_{ux}
• 0.9G + W_{uy}
• 0.9G - W_{uy}
• G + \psi_EQ + [32 different combinations involving +/- 0.1b eccentricities and 100% of the seismic load in one principle direction and 30% in the other}, and
• G + [32 different combinations involving +/- 0.1b eccentricities and 100% of the seismic load in one principle direction and 30% in the other].

218. Then there are numerous combinations required to be checked for short term, long term and creep deflections under gravity, wind and/or seismic loading.

219. Plus, in many instances, the fire load condition, where reduced dead and live load combinations must be assessed for a structure weakened and softened by fire.

220. For industrial structures with a lot of plant (Treasury hasn’t destroyed all of our productive base yet), there will be numerous additional gravity load cases to consider, and most likely numerous crane load case combinations, including fatigue loading.

221. And I haven’t even mentioned reviewing shop detail drawings and construction observation, or the additional work caused when mistakes on site have to be sorted out.

222. The present fee structure combined with the lack of skilled staff who can work in a competent, consistent, coordinated manner cannot handle all this. Is it any wonder that all of the shockers I have exposed have occurred? It would be unbelievable if they had not.

223. Any worthwhile findings and recommendations from this Royal Commission that do not address the fee and skill base in structural engineering will make this situation worse, if for no other reason than the fact that proper diaphragm design must finally be addressed, and will create a massive additional workload if it is to be done properly.

224. Design Features Reports and Engineering Peer Reviews
I believe it has been put to the Royal Commission that if all projects had an initial engineering Design Features Report, that would go a long way towards ensuring ‘quality design.’ I disagree – virtually all of the major shockers in my Open Letter and since had Design Features Reports, and what good did that do?

225. Similarly, there seems to be an obsession with Peer Reviews, both internal (which I shall deal with soon) and external to a design firm. These are worth nothing, because the peer of an idiot is an idiot. We need ‘Significantly Better Than Peer Review’ in most cases. Nearly all of the major shockers in my Open Letter and since had peer reviews for Africa, both internal and external. There were ‘leading firms’ and ‘Fellows of IPENZ’
all over the place, and the reviews counted for nothing in most instances. And the reason was that the peer reviewers were the product of the same ‘system’ as the designers, with the same preconceptions, closed minds and mantras. In many instances, peer reviews should be a two stage process – one early on, at the ‘butter paper, thick marker’ concept stage, and then at the end.

226. I know some highly frustrated and very tired first rate engineers who pick up a lot of bad things in the peer reviews they do, but there are far too few skilled engineers like them nowadays. If you read my Open Letter closely, you will see many of the same (disguised) identities appearing several times, first designing a shocker, then later peer reviewing and approving one, faults and all. Many Producer Statements PS2 – Design Review are no better than many of the PS1’s and PS4’s – not worth the paper they are written on.

227. In-house Checking and ‘Quality’ Within The Engineering Profession, and ISO 9001
“A book that is proof read by two people is not proof read at all.” An excellent saying, and so true. In many instances, very sloppy designs are done, especially in large firms, with the ‘designer’ hoping that the internal checker will find all the problems. But when it comes time for the check to be done, the attitude of the management and often the designer is “You’re just checking it, not designing it.”

228. My experiences doing in-house checks while working on contract in large firms have usually been terrible. All too often I was presented with clearly deficient but otherwise completely unintelligible design calculations and drawings, and then told that I was checking it, not designing it.

229. A typical example of this rubbish related to the use of finite element programs to analyse complex two way foundation slabs for industrial type structures. Instead of being presented with a logical sequence of calculations showing the development of the loads and the finite element model, and clear output, along with a proper assessment of the true moments incorporating $M_{xy}$, I would be confronted with two completely unintelligible plots of $M_x$ and $M_y$, no check as the the accuracy of the analysis, and some scribbled ‘calculations’ for the design. And almost certainly no assessment for punching shear around the columns that involved unbalanced loads and moment transfer to the columns.

230. As Shewart, Juran and Deming showed to the Japanese after the Second World War, true quality is built in from the start, at every stage. It becomes part of a process of continual improvement, and through the elimination of waste and rework, produces real productivity gains. Quality cannot come from inspection. Typical of Japanese manufacturing, light bulb manufacturers would make as many light bulbs as quickly and as cheaply as they could, and then inspect them prior to packaging. One third of the product had to be discarded. The three Americans pointed out that this represented massive cost and waste, and showed how much additional production was required to generate the profits to pay for this waste. The rest was history, until the banksters brought down the Japanese economy in share and land price bubbles.

231. Along with the cults of management and leadership that supplanted engineering in the consulting firms, ‘quality,’ in the form of ISO9001 was introduced, largely as a marketing exercise. The less said about this the better, but at most it constitutes worthless ‘box ticking,’ and is the antithesis of what Shewart, Juran and Deming spent
their whole working lives developing and promoting. I was heartened in one firm I worked at when ISO 9001 was introduced. The most vocal critics were the best practising engineers. Most of the worst shockers in my Open Letter were from ISO 9001 certified ‘quality’ companies.

232. So much of the ineffectual legislation and ‘reform’ since the Building Act 2004 seems to be predicated upon the idea that ‘supervision’ and ‘inspection’ by skilled supervising staff will prevent most site problems. This is nonsense. We don’t have the skilled staff to stand around supervising and inspecting, and often untrained and incompetent workers do damage to work already done, such as mindless cold rebending of starter bars, at the same time they are making a balls up of the actual work they are doing, which then has to be redone.

233. Dynamic Fracturing of Reinforcing
I must raise this here, to use in the next section. During one of his appearances before the Royal Commission, Alan Reay used the term ‘strain hardening’ erroneously to describe an ‘unexpected’ fracturing of reinforcing bars in concrete shear walls, beams and columns that has led to many buildings having to be demolished. Richard Fenwick corrected Reay, and described the problem as one due to excessive concrete strength in lightly reinforced members. Instead of the distributed cracking that one expected to see at the ends of shear walls, beams and columns, only one wide crack would form, forcing all of the elongation of the reinforcing to occur over a very short length. This led to excessive localised strains, and subsequent reinforcing bar fracture.

234. But this does not appear to be just a function of high ‘normal’ concrete strength and low reinforcing content. Concrete stiffness and strength varies significantly, depending on the rate of loading. High rates of loading increase the Young’s modulus (stiffness), compression strength and tensile strength of concrete, compared with slow rates of loading. John Mander mentioned this in relation to the rate at which concrete test cylinders are loading in testing machines, and well established methods for assessing vibrations in concrete and composite slabs take account of the increased Young’s modulus of concrete under dynamic loading.

235. It appears this increase in dynamic strength of concrete has played a major role in the ‘unexpected’ fracturing of reinforcing, except that this was not unexpected, or should not have been. Attached in Appendix 16 is a paper titled ‘Effect of Loading Rate on Anchorage Bond and Beam-Column Joints’ by Chung and Shah, ACI Structural Journal, March-April 1989. Figs. 2, 8 & 16 clearly show a transition from distributed cracking to concentrated cracking as the rate of loading increases.

236. The low rates of ‘seismic’ loading that have by necessity been applied to the numerous reinforced concrete wall, column and beam assemblages that have been tested over the years throughout the world have not accurately reflected real dynamic response. This poses a major problem for seismic resistant reinforced concrete design, especially of the conventional kind, but the question has to be asked as to why the reinforced concrete academics, researchers and industry representative in NZ were not aware of this effect, given that the aforementioned paper (and there must have been others like it) is 23 years old?
J. Roles and Responsibilities – Part 3 – ‘Key Players’

237. **Ministry of Business Innovation and Employment**
This ‘super-ministry’ was recently created through a merger of the Ministry of Economic Development, The Ministry of Science and Innovation, the Department of Labour (which I think included Immigration NZ), and the Department of Building and Housing.

238. This merger was conceived of and completed while two of these departments were subject to scrutiny by two separate Royal Commissions, the one enquiring into the Pike River mine disaster, and this one. I cannot see how this ‘restructuring’ of these departments did not breach constitutional convention.

239. I have explained at length above why the Ministry of Economic Development, and the like minded Treasury and Commerce Commission must have absolutely nothing to do with the building industry in NZ. The damage they have done is almost irreparable; their continued involvement can only cause more damage.

240. I believe the Ministry of Science and Innovation (MSI) was previously given a large part of the Building Levy and direct tax money that was meant for building research, and the MSI would then pass it on, much of it presumably through GNS as described in my previous submission on GEN.CERC.0003. The MSI knows nothing about the building industry, and should have nothing to do with it. Any ‘handling of monies’ can only add an unacceptable layer of expensive but ineffective bureaucracy.

241. The Department of Labour has clearly been unable to train the workforce the NZ building industry needs. Immigration NZ has been an absolute disaster. Tens of thousands of *completely unskilled* people have flooded into this country every year for decades. Many have ended up in the building industry, and have made a bad situation worse. Some have ended up as ‘carpenters’ who cannot hang a door, or ‘builders’ who do not know what a damp proof course is, or ‘reinforcing workers’ who do not know that spirals must be anchored, even when they are forced to do rework on site, time after time after time. Many of the supposed ‘qualified building professionals’ are no better. Some of the ‘engineers’ who have come in know *nothing*, and are then completely irresponsible on top of that. Some of the ‘draftsmen’ cannot even work as competent ‘tracers.’

242. The NZQA has exacerbated this situation, by recognising qualifications from countries with the most appalling construction industries, and not applying any additional checks. In addition, they have overseen the *continued* serious decline in standards of building education and training in NZ.

243. I shall get onto the Department of Building and Housing in a moment, but I rightfully condemned it at length in my submission on GEN.CERC.0003.

244. Not only is the Acting Chief Executive of MBIE ex-MED and ex-Treasury, the MBIE website states that there is an Acting Chief Executive for the Department of Building and Housing. This person apparently has an “extensive background in change management.” Is he an engineer? No. Is he an architect? No. A carpenter? No. A draftsman? No. A quantity surveyor? No. The first 10 years of his civil service career was spent in the Department of Social Welfare.
245. One would think that this restructuring would have put the worse than ineffectual Minister for Building and Construction, Maurice Williamson, out of a job, but no. He’s still there, one of 13 ministers MBIE is now ‘responsible’ to! Focus and efficiency indeed.

246. For simplicity, I shall continue to call the MBIE ‘Building and Housing Group’ the Department of Building and Housing (DBH), because their website and documents haven’t been ‘rebranded’ yet. (And how much is that going to cost?)

247. As I clearly showed in my submission on GEN.CERC.0003 (especially in my complaint to the Auditor-General), the conception of the DBH was completely wrong, it is not ‘fit for purpose,’ and has not only failed to address the crisis in the construction industry (apart from the worst leakers), it has refused to even acknowledge this crisis, even as the stadia and other roofs continue to fall.

248. The DBH was supposed to advise “the Minister for Building and Construction on matters relating to building control.” Appendix M of my submission on GEN.CERC.0003 contains the DBH’s briefing to the incoming Minister, following the 2008 General Election. Despite the Waitakere Trusts Stadium failing during erection under its selfweight, the near total collapse of the Vector Arena, and all the other shockers that had occurred since my Open Letter, there was no mention of any problems with structural engineering in this briefing. Even ‘leaky buildings’ were treated as a relatively minor, historic problem. (I covered what was required to get the DBH to face the reality of that disaster in my submission on GEN.CERC.0003).

249. Williamson wouldn’t have acting on such proper advice anyway, but the 2011 briefing to the incoming Minister was even worse. No mention of even Stadium Southland, not the appalling performance of so many of the modern buildings in the Christchurch CBD, buildings which developed all sorts of unacceptable catastrophic collapse mechanisms, building that would have collapsed if the 22 February 2011 earthquake had been less intense, but lasted for a minute or a bit longer.

250. The DBH was also aware of the setting up of my IPENZ Structural Taskforce, and its subversion by the IPENZ Deputy President and the Chief Executive, a food technologist, but the DBH didn’t care.

251. The staff of the DBH consume many tens of millions of dollars of Building Levy and taxation a year on their own salaries, but how many have any experience whatsoever in the building industry? Very few. I think they employ two structural engineers.

252. The technical capability is so ‘light’ that when Stadium Southland collapsed, investigators had to be brought in from the private sector. Many more buildings than the ‘big four’ in the Christchurch CBD should have been investigated, but for the investigations of the CTV, PGC, Grand Chancellor and Forsyth Barr buildings, the DBH was forced to “go to the market place.” Most of the companies they invited tenders from had designed or approved the shockers exposed in my Open Letter or since then, and most of these were seismic shockers. Of course, I was not invited to tender, because I am not a ‘recognised expert.’ Thank God for that.

253. Hopefully, Carl O’Grady will be called to appear before the Royal Commission. When he does, ask him about his conversations with DBH staff when he enquired as to when the reports on the Stadium Southland collapse would be released.
254. If the DBH was a building, it would be condemened as unfit, demolished, and dumped in a controlled tip. The DBH needs to be demolished, and replaced with an effective technically competent body that will finally fulfill its fiduciary duty to the people of New Zealand.

255. Defining proof of the DBH’s complete incompetence can be found in Appendix 17, in the form of the Order in Council ‘Building (Designation of Building Work Licensing Classes) Order 2010,’ made by the Governor-General on 1 March 2010.

The fundamentally flawed and ridiculously named Licensed Building Practitioner scheme formed a major part of the Building Act 2004, and the Department of Building and Housing has been working on the scheme since its inception.

I have stated on numerous occasions that the Licensed Building Practitioner (LBP) scheme as envisaged and implemented will not and cannot work, because it has already failed completely and utterly.

To all intents and purposes, the LBP scheme has been in full operation since 2003, in the form of the ‘chartering’ of professional engineers pursuant to the Chartered Professional Engineers Act 2002. Chartered Professional Engineers are automatically licensed as Category 3 Design and Category 3 Site LBP’s.

Nearly all of the structural shockers that have occurred since my Open Letter have been designed, closely supervised and/or approved by Chartered Professional Engineers.

Despite having spent years, and many tens of millions of dollars of taxpayers’ money on the LBP scheme, they have made a complete mess of defining the building categories.

What was intended for the building categories (not that I agree with them) is:

- Category 1 – simple single dwellings, which can be designed by ‘design draftsmen,’
- Category 2 – more complex dwellings, small apartment complexes, etc, which can be designed by ‘intermediate’ designers, and
- Category 3 – buildings that require specific design by, or under the close supervision of, a Chartered Professional Engineer and/or a Registered Architect.

(‘Buildings’ should actually be ‘structures.’)

Firstly, this Order in Council, like almost everything else, wrongly concentrates on ‘leaking building envelopes.’

Hence, a ‘risk score’ regarding the risk of leaks in the building envelope must be determined. The number of storeys affects the risk score.

Excluding ancillary buildings and outbuildings, on page 7 of the Order, a category 1 building is defined as a ‘sleeping single home’ with a risk score less than or equal to 12.

A category 2 building is defined as neither category 1 nor category 3.
A category 3 building is defined as not a sleeping single home, whose building height is or exceeds 10 metres.

Building height is explicitly defined as one thing, and one thing only – ‘the vertical distance between the upper surfaces of the floors of the building’s lowest and highest storeys.’

In other words, structures like the Tiwai Point aluminium smelter buildings and the NZ Steel mills, which are single storey buildings well over 10m height, and which support massive cranes, will be Category 2, not 3.

The main roof of the Vector Arena and the Waitakere Trusts Stadium, which are single storey structures well over 10m height, and which seat thousands of people, will be Category 2, not 3.

According to this, if we had a nuclear reactor, the main containment vessel, and the main generating hall, would be Category 2, not 3.

There is also no consideration of the number of occupants or the post-disaster function of the building. These are meant to be governing aspects of the design process. But perhaps worst of all, buildings are in reality only a subset of structures, but the DBH seems incapable of grasping that fact, especially at the highest levels.

Utter nonsense, and the taxpayer is paying for it.

256. Building Advisory Panel
One could be excused for thinking that either this panel is not providing the correct advice to the DBH, or the DBH is not listening. In fact, both are correct.

257. One layer down, at least three advisory groups were set up soon after the formation of the DBH; one for disabled access, one for ‘leaky buildings’ and one for structures. Phil O’Sullivan got invited onto the one for ‘leaky buildings,’ but of course I was not invited onto the one for structures. Which is good, because I couldn’t have stood it for long. Despite there being a few good individuals on it, the structural group mainly consisted of people from companies that had designed the shockers described in my Open Letter, or similar shockers since. Even then, the group gave some good advice which the DBH never listened to, or acted on so slowly there was no effective difference. Some of the better people just gave up and quit.

258. Territorial Authorities/Building Consent Authorities
I have pretty much covered these in my earlier submission and above. They have been stripped of their technical competence, and were then expected to ‘carry the can’ for so many of the shockers, but are now being manoeuvred out of being ‘last man standing’ with regard to housing shockers. The farce is that for anything other than a house, flat or apartment building, the BCA grants the Building Consent, but the courts have determined that the BCA has no liability regarding that consent or the subsequent Code Compliance Certificate. The whole system is crazy.

259. Licensed Building Practitioners
A total farce – box ticking applied to a competence starved industry, and highly restrictive box ticking at that. As said above, the complete and utter debacle of Licensed
Building Practitioners in the form of Chartered Professional Engineers shows the whole thing is a waste of time and money as conceived and implemented.

260. **Standards New Zealand**

In the 1990’s, all public funding was withdrawn from Standards New Zealand, and it has since been forced to survive as a purely commercial entity. It would have been better for the construction industry if Standards New Zealand had been done away with completely when its public funding was withdrawn.

261. One complete and utter fallacy that engineers in NZ cling to is this belief that a ‘Standards’ body must control the development of ‘codes.’ This is absolute nonsense. In Britain, Australia and NZ, the respective national standards bodies have traditionally performed this role, but in the main centres of earthquake engineering expertise, namely California and Japan, the complete opposite is true.

262. In the US, various loadings, design and materials ‘codes’ are adopted on a federal, state, county and city level. Typically, loading codes have been written by the likes of Building Officials and Code Administrators International, Inc. (BOCA), International Conference of Building Officials (ICBO), and the American Society of Civil Engineers (ASCE). Steel design and construction codes typically adopted have been those of the American Institute of Steel Construction (AISC), while the American Concrete Institute (ACI) handled reinforced concrete. Building material manufacture and testing has invariably been codified by the American Society for Testing and Materials (ASTM). In California, seismic design is usually governed by the Structural Engineers Association of California (SEAOC) ‘Blue Book,’ – Recommended Lateral Force Requirements and Commentary.

263. Similarly in Japan, the Architectural Institute of Japan, which deals with architecture, planning, and the structural design of buildings, has produced widely used structural codes for the design of steel, concrete, masonry and composite structures. The 1979 version of the AIJ Design Standard for Steel Structures was used to design most of the Hot and Cold Mills for the NZ Steel Stage 2 expansion.

264. It has reached the point where having ‘Standards’ in charge of ‘the codes’ is as farcical as having bureaucrats in charge of ‘the industry.’ I stand to be corrected, but as I understand it, the British representative on the committee writing the Eurocode structural steel design code was a Standards ‘bureaucrat,’ and not a structural engineer at all, let alone an expert in steel design and construction.

265. With the odd exception, the ‘stewardship’ of structural codes in NZ is in crisis, and these codes need to be taken off Standards NZ. The quality of any particular standard, as regards actual content, presentation and ‘errata,’ appears to be completely dependent on the people in control of the particular committee that wrote that standard. ‘Standards NZ’ is completely irrelevant, other than being a very expensive printer.

266. General problems have been ‘code bloat,’ massive price increases, standards being reviewed at the whim of Standards NZ, and not the needs of the industry, and some standards with appalling proof reading, with numerous errata and corrections required.

267. Despite their obvious flaws, NZS 4203:1984 and NZS 3101:1982 were of a manageable size, just like AS1250 and the AIJ steel code of 1979. These were reasonably logical,
and could guide the structural engineer along the right path (shear wall protected non-
ductile gravity frames aside).

268. The latest loadings standard, AS/NZS 1170 consists of five parts for NZ and five
commentaries, and is many times larger than NZS 4203:1984. The cost of this complete
code, for one paper copy, is now close to $1,000 plus GST. It is absolutely riddled with
a whole raft of variable load factors for live load, depending on what other load case is
involved, yet the net effect of this on building economy is near zero. And for a tall
building in Christchurch, NZS 4203:1984 provides far more accurate seismic horizontal
force coefficients.

269. I get onto NZS 3101:2006 later, but this code and commentary has ‘bloated’ from 280
odd pages in 1982 to almost three times that. NZS 3101:2006 does include some very
useful improvements, but some blunders as well, and it needs to be completely re-
written.

270. The US President Harry Truman had a sign right at the front of his desk that said “The
buck stops here,” and he meant it. He was ‘The President,’ and was prepared to take the
responsibility that went with it.

271. Compare this with the numerous august bodies covered in this discussion paper and the
previous one, which together claim to represent and lead the structural engineering
profession and the construction industry, but take no responsibility for all the shockers I
have exposed. These august bodies are wrong either way – they are either irrelevant to
the implementation of good practice, or they are relevant, have failed demonstrably, and
need to be held to account. This applies to Standards NZ as well.

272. How could Standards NZ allow the setting up of a committee for what became NZS
3101:1982 that allowed Section 6.4.7 to be put in the standard, a section that allowed
‘shear wall protected’ columns to have spirals and stirrups as small as R6 and at as much
as 288mm centres in multi-storey buildings, given the performance of columns at the
Olive View Hospital in the San Fernando earthquake of 1971, and the two photographs
on page 125 of ‘Reinforced Concrete Structures’ by Park & Paulay, 1975? (Refer
Appendix ζ of my submission on GEN.CERC.0003). Most if not all of these 1980’s
‘shear wall protected gravity frame’ buildings have been demolished, despite apparently
“leading the world in seismic design.”

273. How could Standards NZ allow the setting up of a committee for the loadings standard
NZS 4203:1992 that allowed one man to ‘mangle’ the critical section on Parts and
Portions, such that instead of being improved where needed, the clear and logical section
in NZS 4203:1984 was deleted, and replaced by something unintelligible.

274. Further to NZS 4203:1992, the very clear factored load case combinations of NZS
4203:1984 were done away with, creating uncertainty, and introducing load factors such
as 1.4 for retained soil that local authorities refused to accept.

275. Further to NZS 4203:1992, how could Standards NZ allow this committee, or more
particularly as I understand it, the GNS members of that committee, to completely
ignore the report on Canterbury seismicity prepared under Don Elder by Royds Garden
for EQC in 1991, which proved to be a ‘perfect script’ for the 22 February 2011
earthquake, in their belief, which ignored major earthquakes of the 19th Century, that the
‘big one’ to hit Christchurch would originate from the Main Alpine Fault.
276. Given that assumption, how could the GNS people on that committee then impose the ‘El Centro’ shaped response spectra curves for Christchurch, when the very deep layers of alluvial deposits made response spectra curves more like ‘Mexico City 1985’ much more applicable?

277. These erroneous assumptions, overlooking a report that had to be proved incorrect to be ignored, meant that the horizontal earthquake coefficients for taller, longer period structures were massively underestimated, and reduced significantly from NZS 4203:1984. At a first period of 2.8 seconds, because the Hotel Grand Chancellor was designed to NZS 4203:1984, the 22 February 2011 earthquake was exactly a ‘code level event’ for that building.

278. As if that was not enough, in AS/NZS 1170.5:2004, the ‘seismicity experts’ on the Standards NZ committee proceeded to further reduce the design base shear coefficients for taller, long period Christchurch structures designed to be limited ductile (μ=3) or fully ductile (μ=6) by about an additional 25%! (Refer Appendix δ in my submission on GEN.CERC.0003).

279. How could Standards NZ allow the completely error riddled structural timber standard NZS 3603:1993 to be released, then never updated for the next 19 years, other than to change the allowable timber stresses to reflect the weaker and softer modern Radiata Pine?

280. Leaving the murky disaster of untreated timber alone, how could Standards NZ allow a committee to be formed that removed from the non-specific light timber framing standard (which is used for most house construction in NZ) NZS 3604:1999 the absolute requirement for a damp proof course (DPC) to be installed under all internal timber (more than 300mm away from the exterior) in contact with concrete or masonry?

281. How could Standards NZ have allowed a committee to be formed for NZS 3101:1995 that deliberately ignored my submission on precast floor units, when the detail I asked to be made mandatory was eventually adopted over a decade later, at least for hollowcore, after the construction of many hundreds of unnecessarily deficient buildings in the meantime?

282. Given the ‘industry wide expertise’ on this committee, how is it that it either ignored or did not know about the very significant effect that rate of loading has on the crack patterns and hence ductility demands on reinforcing bars in seismic resistant reinforced concrete structures? This is the critical issue as to whether reinforced concrete is suitable for seismic resistant structures, other than low squat structures. The NZS 3101:2006 committee did not appear to address this either.

283. How could Standards NZ have allowed the complete and utter debacle that was the drafting of the combined Australia/New Zealand concrete reinforcing standard AS/NZS 4671:2001, and the introduction of Grade 500E reinforcing into the NZ ‘marketplace?’ This absolute disaster still blights the industry. It should have been addressed immediately, but Standards NZ did nothing. What is worse, nearly all of the ‘leaders’ of the profession and the industry have treated this rubbish as ‘the writ of God.’ It should have been thrown out and rewritten over a weekend; instead, it still blights us, with no salvation in sight.
284. First, the drafting of the combined standard. Such nonsense is driven by the likes of the Commerce Commission and Treasury, when the Australian and New Zealand reinforced concrete sectors are enormously different, if for no other reason than seismic design. One or two NZ representatives were on this committee, and one of them stated what I am about to say quite openly at a seminar introducing the new standard and the new reinforcing grade.

285. This person stood up in front of 40 to 60 engineers, and said that of the grades of reinforcing covered by AS/NZS 4671:2001, it was only ever intended that three would ever be allowed to be used in NZ: low ductility welded wire mesh, Grade 300E (mild steel) and Grade 500E MA (micro-alloyed). Then he said “But the relevant clauses never got put in the standard” !!!!!!!!!!!!!!!! Either he stuffed up, or he was overruled by the Australian majority. Either way, the nonsense that is AS/NZS 4671:2001 should have been thrown in the rubbish bin, and the existing Grades of 300 and 430 retained.

286. [For your information, most modern structural steel produced through controlled rolling of continuously cast steel has material properties that we want in seismic resistance structures. However, most reinforcing steel produced around the world does not have the properties we desire for seismic resistant reinforced concrete structures. That is just the way of things – it is not a ‘restraint of trade’ issue. The grades mentioned above consist of a number followed by an ‘E.’ The number represents the nominal yield stress in megapascals, and the ‘E’ represents a ductile seismic grade where not only the minimum yield stress is carefully controlled, but also the maximum yield stress and the elongation or ductility of the reinforcing. The ‘MA’ stands for ‘micro-alloyed,’ because the increase in strength from 300 MPa to 500 MPa is achieved by the addition of relatively expensive metals such as manganese, instead of the less desirable quenched and self-tempered process (QT) and unacceptable cold worked process.]

287. As a result of the omission of the required clauses in AS/NZS 4671:2001, all sorts of lower ductility QT steel and God knows what else flooded into the country, to the point where Pacific Steel was forced to introduce a 500E QT product to compete. But it gets worse.

288. At this seminar, the same person was asked what the Californians and Japanese used for their ‘seismic reinforcing.’ He said he did not know! Sensible people in the audience were stunned. It turned out that the Grade 500E MA reinforcing was developed by one person using the non-seismic European type of reinforcing as a starting point, in an attempt to improve the ‘competitiveness’ of reinforcing steel, without any reference to what the Californians and Japanese did. But it gets worse.

289. As described in my earlier submission, several months after this seminar, a major story in the NZ Herald covered the fracturing of Grade 500 reinforcing under cold rebending in the appallingly conceived and designed tilt up panel buildings that now blot the landscape. But what about all the bars that didn’t quite break on rebend? How will they perform in an earthquake?

290. The QT steel that has poured in, much of it from dubious sources, is not allowed to be cold rebent, nor can it be hot rebent if it is to keep its strength. It should not be welded, yet how are the spirals to be anchored if the reinforcing fabricators refuse to use hooks?
291. Even the Grade 500E MA should not be welded because of the near impossibility of doing it right in the field, and any stray tack weld onto either a Grade 500E MA or QT longitudinal bar can lead to subsequent brittle fracture.

292. Get Standards NZ to justify all this.

293. When I was on the SESOC Management Committee, and wrote a feature article in the SESOC Journal saying this was unacceptable and it was up to competent NZ structural engineers to determine what reinforcing was allowed to be used in this country, and no one else, one member of that Management Committee was terrified that if SESOC’s name was attached to the article SESOC would be sued by the importers of this rubbish, and another member of the Management Committee, himself a member of the NZS 3101:2006 committee stated that I formed a ‘fanatical fringe.’

294. One thing I find unacceptable has been the near complete refusal to acknowledge or properly codify precast concrete construction in NZS 3101 over the decades, despite the dominance that precast concrete, and especially precast concrete floors have in the deskillled NZ construction industry. The floor units themselves, which are one way slabs, often violate the perfectly reasonable detailing requirements for cast in-situ one way slabs. As typically designed and installed, such precast floor units must induce at least primary torsion on at least perimeter concrete beams, torsion that cannot be resisted under seismic loading if the ‘theory’ of fully ductile reinforced concrete ‘weak beam – strong column’ behaviour is valid. In addition, these precast floor elements must introduce all sorts of discontinuities and reductions in the effective width of the supporting members, but the standards ignore this, and as far as possible, treat everything as cast in-situ, when it is not.

295. The committee set up to write NZS 3101:2006 contained many of those one would expect to be there for no other reason than the organisations and entrenched interests they represented. Despite the detailed content of my Open Letter, of course I was not invited to be involved. I cannot remember whether it was in direct response to the content of my Open Letter, or an initial submission I made, but I received a derisory response from the committee, or more likely, the clique that controlled it.

296. In response, I contacted several top engineers, and made my concerns known. I asked for a meeting to see what could be done. One of them, unfortunately, demanded I write my concerns down, to be reviewed by some genuine concrete experts before the meeting. I quickly dashed off 24 pages, and these were reviewed. Only one point was challenged by the experts. I then met with two top engineers, and one senior representative of ‘the engineering establishment.’ They all agreed that I had valid concerns, and were interested in sorting out the concrete standard. The ‘establishment figure’ said he would contact a senior person at Standards NZ in person; he was not prepared to say what he had to say over the phone, or put it in an e-mail or on paper. He duly met the senior person at Standards NZ, and was told that ‘the standard is a work in progress,’ and that was that.

297. I have not wasted my time counting the pages because they are numbered by section only, but I think the two parts of NZS 3101 (‘code’ plus commentary) total 750 pages. The committee were only given 7 days to proof read these 750 pages.

298. NZS 3101 and its successors are based on the ACI concrete design code ACI 318. In NZS 3101:1982 and NZS 3101:1995, the torsion sections of ACI 318 were faithfully
reproduced. But in NZS 3101:2006 they were mangled, with lots of stupid basic mistakes, and that has only now been sorted out.

299. One thing I specifically asked to be codified was the effective width for shear in beams supporting precast beams. Shear stresses must increase as any part of the width of a beam reduces – that is Mechanics and Properties of Materials 101. Not only must most precast floors ‘notch’ the sides of the supporting beams, with the introduction of the ridiculous ‘low friction bearing strips,’ there must be such notching, and a reduction in beam shear strength. Either the cast in-situ shear provisions are far too conservative, or many beams supporting precast are overstressed and not code compliant.

300. My requests for the effective width of beams supporting precast floors to be properly codified were repudiated with ridicule. But look at the pictures defining in-situ beam width in Figure C5.A5 on page C5-14 of Appendix A to C5 in NZS 3101:Part 2:2006 (Appendix 18). This section is straight out of the Eurocodes, I think, and was put in for commercial reasons, to increase the market opportunities for steel fibre reinforced beams, something that is last on the list of critical issues that need to be addressed regarding reinforced concrete in NZ. It was all right to let such a diagram into the standard to promote commercial interests, but not a similar diagram that would hurt commercial interests.

301. I could go on but will stop there, except to note that this standard should have banned non-ductile mesh reinforcing outright, and it did not. As a result, hundreds of more buildings that will suffer unnecessary damage under seismic loading have been built.

302. One practical issue with NZ standards nowadays is that they are invariably in a non-serif Arial-type font, which human beings find very hard to read when more than a few words are put together (all novels and film scripts are printed using serif-type fonts, such as the one I am using), and printed on glossy white paper, the worst thing possible given the harsh cheap fluorescent lighting most structural engineers have to work under.

303. **BRANZ**

   It isn’t what it used to be.

304. **The New Zealand Construction Industry Council (NZCIC)**

   The Discussion Paper states that “the NZCIC is able to take a sectoral approach to matters and operates on a consensus basis.”

305. The latter implies the erroneous fallacy of the argument *ad populum*. With regards to the important steel support platform that was overstressed 3000% under the design seismic loads in my Open Letter, what is the ‘consensus’ between this and a correct design? 1500% overstress, of course, a consensus that is still completely and utterly unacceptable.

306. Who makes up the NZCIC? The membership list off their website, with the contacts for each member organisation, is included in Appendix 19. There are 31 member organisations.

307. The tree is known by its fruit. So is the forest. This ‘forest’ of organisations contains some good and responsible organisations that are key to turning the crisis around. Other organisations are just the opposite, like IPENZ.
308. As a whole, this ‘forest’ can reasonably be considered to be the New Zealand building industry. Therefore this ‘forest’ is responsible for the ‘leaky building, rotten timber’ disaster, and it did nothing to stop it. The individual members of some of these organisations conceived of and supplied the defective methods and materials, and others designed and built the shockers. A $22 billion dollar disaster. And who had to stop it? Two men, Greg and Phil O’Sullivan, although I note Phil being the contact for the building surveyors.

309. Similarly, many of the members of the organisations making up the NZCIC designed the structural shockers I have exposed, or approved them, or built them.

310. Others like ACENZ and the DBH have done nothing effective to address the crisis, while IPENZ has actively subverted reform.

311. The NZCIC is, at best, a complete irrelevance with regard to effective reform, which must come soon before it is too late.

312. The Remainder of the Discussion Paper
   I have pretty well covered the remainder of the Discussion Paper above, or in my submission on GEN.CERC.0003, and will leave it at that.

313. The Insurers
   In 2008, I had pinned hopes that the insurers, through the Insurance Council, could enforce some reform. It is fundamental that all mortgaged buildings, most homes and ‘public’ buildings, and all commercial buildings, must be insured. If the insurance companies turned around and said that in order to have insurance, the front door of a building must be painted pink, then nearly every front door in the country would be painted pink.

314. I met with representatives of the Insurance Council in Auckland in late July or early August 2008, and they took interest in the problems I was raising. I then asked if buildings that could be shown to have been designed and constructed properly, in strict accordance with sound engineering principles and not the nonsense that passes for standard practice today, could be rewarded through lower insurance premiums. They said that the NZ insurance market was purely ‘cost driven,’ with massive reinsurance offshore, and such a scheme could not be introduced.

315. They may be reconsidering that now, but have not got back to me. If what I heard on the radio was correct, one insurance company recently got out of the NZ market because they said that last year, they took in $30 million in premiums, but paid out $300 million in earthquake damage, and that was “not a sustainable business model.” But it’s been 80 years since the previous big earthquake in a NZ city – that’s 70 years of little or no claims. What did they do with all the premiums?
K. Some Additional Comments

316. Novation
Unlike some of the very large overseas contracting companies, NZ contractors do not have significant research and/or design departments. Some of the larger firms obviously have some design capacity for falsework and other temporary works, and employ engineers who are capable of designing buildings, but, as a general rule, contractors in NZ are completely dependent on consulting engineering firms for the design and drafting of the buildings they construct.

317. One practice on large projects that has started to occur, and which I am opposed to for a variety of reasons, is that of the structural engineer and other design professionals being ‘novated’ to the contractor after the initial design phase. This tends to be done on projects done on the basis of ‘a guaranteed maximum price.’ This practice further erodes the capability of the structural engineer to perform to the highest standard. Until such time as NZ contracting firms are sufficiently ‘upskilled’ that they can do cast in-situ construction to match other First World countries, and have raised their productivity to match firms in other First World countries, and spirals are anchored properly with hooks on all projects without second thought, and starter bars are no longer mindlessly cold rebent, then ‘novation’ should be banned.

318. As outlined above, it is the low levels of technical competence, basic skills and sound practice that so badly affect cost and productivity on building projects in this country. Fiddling with the form of contracts, especially when they reduce the status and control of structural engineers still further, does not deal with these fundamental problems; in fact, it makes the problems worse.

319. Let He Who Wins Laugh
The claim that “New Zealand leads the world in seismic engineering” is akin to an army claiming to be undefeated only because it has never fought a war. To any open mind, the performance of modern buildings in the Christchurch earthquakes, the failure of ‘the experts’ to acknowledge what to them should have been the blindly obvious seismicity of that city, together with all of the other structural shockers that have been exposed, and all the other failures of the NZ building industry, this claim to NZ supremacy in seismic engineering is hollow indeed.

320. Unfortunately, this smug claim to supremacy has been accompanied many times over the years with derisory comments about much Californian and especially Japanese seismic practice. In particular, the Japanese were criticised for designing very stiff, strong buildings that would attract large earthquake forces, instead of the ‘flexible frames’ that ‘benefit’ from the drop off in base shears afforded by the ‘El Centro’ based design spectra.

321. Except that just as the Californians have taken to stiff, strong building forms as a result of earthquake performance observed in the 1980’s and 1990’s, intelligent clients and designers will be using buildings for the rebuild in Christchurch that are as stiff and strong as practicable, to limit non-structural damage, and reduce the risk of having to demolish the building after another strong shake.

322. Will the current ‘leadership’ of the profession in NZ front up, and apologise profusely to the Californians and especially the Japanese for not only ignoring but actually criticising their best practice for decades? I don’t think so.
323. **Codes versus Commentary**
   A great deal of what I can only describe as nonsense has been said and written about “the code is mandatory” and “the commentary is not,” and that “commentaries and guides have no legal standing.” That is not the legal standing as I understand it, or at least what it should be.

324. I have always understood that an engineer is required to apply ‘best practice.’ Clearly, an engineer cannot be expected to know the latest research findings, but sound engineering practice must come first, with the codes used as an aid in quantifying these fundamental principles.

325. As I have explained at length above, numerous fundamental aspects of structural behaviour are missing from many codes and standards, the direction of gravity being just one of them, but if a structural engineer does not understand them and does not apply them, then he/she is not a structural engineer.

326. Commentaries to codes and standards are clearly meant to be read *with* the code, not treated as something to be ‘disobeyed.’ Similarly, codes cannot be compendia of all knowledge, and a responsible engineer can only design many complex elements through reference to guides, textbooks and other information that can never be included in basic structural codes.

327. Constant ‘fiddling’ with codes, their constant reformatting and change of nomenclature must stop. We need codes with sound fundamental practice embodied in them, and which then stay constant, except for critical improvements. Engineers have more than enough to do without having to keep up with constantly changing ‘image’ that lacks substance.

328. **Personal Libraries and Copies of Key Standards**
   What one should see in every office is that each engineer has a large personal library of quality books and design guides that they have read and absorbed, and personal copies of the key codes and commentaries that have clearly been ‘well thumbed’ and absorbed.

329. Unfortunately today, the typical cheap ‘workstation’ in the typical cheap ‘open plan office’ has absolutely no allowance for an engineer to have his/her own library, even if they were of a mind, and instead of each engineer being provided with his/her own copy of the main codes and commentaries, a few copies will be shared around the office.

330. At least when I was a graduate, every engineer had their own, not necessarily legal, copy of the loadings, steel and concrete standards and commentaries. There is nothing as heartbreaking as seeing graduates now, when tasked with a new type of design, taking a copy of a standard they are unfamiliar with, and then ‘groping’ to see which clause in isolation may be applicable to the job at hand.
331. **The Head in the Sand, or Is It Somewhere Else?**

The population of the United States of America is approximately 70 times that of New Zealand. Scaling ‘the shockers’ up to American scale, if, over an 8 year period in the US:

- 140 stadia had been failing under their selfweight, and half of those had nearly failed catastrophically, and
- 70 stadia had actually collapsed under light snow loads they were meant to handle, and
- 70 supermarket roofs and 70 other steel framed roofs had failed because of an incorrectly designed, very common connection detail, and
- A large percentage of the CBD area of the entire country was completely devastated in an earthquake, including vast numbers of buildings claimed to “lead the world in seismic engineering,” and
- Grossly defective and substandard structural products were flooding into the country or being produced domestically, and
- Millions of homes and apartments were found to not only be leaking and rotten but structurally unsound due to design and construction defects unrelated to water damage, there would be “hell to pay.”

332. Executive Presidential Orders and subpoenas would be flying all over the place. The Senate, the House of Representatives and numerous state legislatures would be holding investigations where people would actually be grilled at length, and their testimony recorded, lawsuits totaling trillions of dollars would be underway, billions of dollars of research would be underway, and numerous people, and especially those in positions of responsibility and with a fiduciary duty to protect the public, would be facing long jail terms. Arrests would have been made, notices served, and people ‘struck off.’

333. What happens in NZ? The Minister responsible says “New Zealand has amongst the highest building standards in the world,” and the Prime Minister doesn’t want to know.
L. The Solution

334. The structural engineering profession and the construction industry are in absolute crisis. The maintenance of the status quo, with a little ‘tweaking,’ will not address this crisis – it will ensure that total failure occurs soon. Unprecedented action is required, because the present crisis is unprecedented.

335. As a first step, there needs to be acknowledgement of this by Parliament, on a bipartisan basis, an acknowledgement of the critical importance of the building and construction industry, and a clear statement that it is to be saved at all costs. A competent, responsible Minister must be appointed. Building legislation must be immediately redrafted by people who know what they are doing and what is required.

336. The DBH has to be disbanded, and all other government departments have to relinquish any control over the building industry.

337. To regulate and guide the building industry, the professions and trades within the industry, and the training and competence of the people within the industry, a new body must be created.

338. Let us call it ‘The Building and Construction Commission.’ This body would have a small and technically competent permanent staff, and be overseen by a Governing Board.

339. All building and research levies, all taxpayer money allocated to the building industry, including that for research, and all money allocated for university, technical institute or trade training in areas related to the building industry must come under the control of the Commission.

340. Membership of the Governing Board must be by invitation only from the initial Board members, and ‘the usual suspects’ will not be invited. Until the crisis is turned around, **the core of the Board must come from those structural engineers who have identified the crisis, and worked for years for effective reform.** Like minded architects, tradesmen and the like will be invited as well.

341. Under the main Building and Construction Commission would sit the following departments:
   - Structural Engineering,
   - Architecture,
   - Building Services,
   - Geotechnical Engineering,
   - Construction.

342. The Building and Construction Commission will directly (or commission and control):
   - Advise the Government, the Minister and Parliament,
   - Draft relevant legislation and regulations,
   - Write building codes, standards, guidelines and texts, with an emphasis on guidelines so that everyone in a particular field is known to have access to the critical basic information that is required, and be expected to know it and apply it always, *or else,*
• Register engineers, architects, tradesmen, other workers and companies working in the construction sector, and provide swift, fair and effective disciplinary measures where required,
• Reinstitute a genuine apprenticeship scheme for the industry,
• Ensure minimum standards and guide the training at universities, technical institutes and the like, and fund that training,
• Regulate building products,
• Fund and guide an expanded research and testing programme, focused initially on what we need to know now,
• Recruit the best people overseas to make up shortfalls in our skill base, but especially to vastly increase the number of people available to train others at universities, technical institutes and in trade training,
• Extricate NZ as far as possible from all of the ‘free trade’ entanglements as they relate to the construction industry,
• Have control over, and veto power over, any decisions relating to the building sector made by Immigration NZ, NZQA and the importation of building materials,
• Carry out a major education program for building owners and the public, so they can at least make partially informed decisions,
• Reinstate and enforce appropriate minimum scales of professional fees,
• Get proportionate liability introduced, instead of joint and severable liability, backed up by unbreakable insurance schemes that ensure every party can pay if found to be liable,
• Provide technical support to the courts and the like, so that claims for building defects can be assessed properly as to the technical issues involved.
Appendices – Table of Contents

Appendix 1 - Information Sent to the Ministry of Economic Development Relating to the Drafting of the Building Bill 2003

Appendix 2 - New Zealand Building Code: Clause B1 - Structure

Appendix 3 - AS 1250-1981

Appendix 4 - Extract from ‘Steel Designers Handbook’ by Gorenc & Tinyou

Appendix 5 - Extract from ‘Review of Planning and Building Controls’ The Treasury, 1983

Appendix 6 - Article from NZ Herald on Slump in Apprentice Skills, 1 October 2004

Appendix 7 - John Scarry Comments on Proposed NZTA Composite Bridge Design Guide

Appendix 8 - Supermarket Collapse - Burnaby, British Columbia, 1988

Appendix 9 - Getting It Right The First Time - Engineers Australia, Queensland, October 2005


Appendix 11 - Defective Chinese Hollow Sections Exported to the USA

Appendix 12 - Extracts from John Scarry Submission on Building Amendment Bill (No 4)

Appendix 13 - Failed Timber Dance Floor, University of Canterbury, 2012

Appendix 14 - Condemned Apartment Building on Park Terrace, Christchurch

Appendix 15 - Scarry/Clifton Submission on Draft Central City Plan for Christchurch, Sept. 2011

Appendix 16 - ‘Effect of Loading Rate on Anchorage Bond and Beam-Column Joints’ by Chung and Shah, ACI Structural Journal, March-April 1989

Appendix 17 - Building (Designation of Building Work Licensing Classes) Order 2010

Appendix 18 - Figure C5.A5 of NZS 3101:Part2:2006

Appendix 19 - NZCIC Membership List
Appendix 1

Information Sent to the Ministry of Economic Development Relating to the Drafting of the Building Bill 2003
Draft Proposals for the Organisational and Legislative Framework for the Reform of the Structural Engineering Profession and the Construction Industry in New Zealand

by

John Scarry

February 2003
CONTENTS

Contents

Proposed Reconstitution of the Building Industry Authority

Diagram - Long Term Reorganisation of the Building Industry Authority and the Construction Industry

Diagram - Short Term Action Required to Address Worst Aspects of Practice Until Reorganisation in Place

Draft Proposals for the Organisational and Legislative Framework for the Reform of the Structural Engineering Profession and the Construction Industry in New Zealand

Extracts - Peter Miller – Engineers Australia Nov. 2002

John Allen – New Civil Engineer International Jan. 2003

The Structural Engineer – 1 Oct. 2002 (Page 5)

The Structural Engineer – 15 October 2002 (Pages 29-30)

The Structural Engineer – 15 October 2002 (Page 10)

Part of Report on Northridge Earthquake – BNZSEE - 1994

Northridge – Questioning Our Codes – Civil Engineering June 1994

One Unacceptable Reality of ‘Free’ Trade
Draft Proposals for the Organisational and Legislative Framework for the Reform of the Structural Engineering Profession and the Construction Industry in New Zealand

by John Scarry – February 2003

1. Introduction
This set of notes is a draft of the organisational and legislative framework that is necessary to achieve reform of the structural engineering profession and the construction industry in New Zealand.

The underlying concepts are predicated on the actual state of the profession and the industry, as outlined in my ‘open letter’ to IPENZ, and an understanding of the entrenched attitudes, rigidity of thought and economic pressures which pervade both the profession and the industry.

For simplicity, the proposals concentrate on the structural profession and the directly related sectors of the construction industry, but would obviously need to be extended to cover all other aspects of construction governed by The Building Act.

The proposals may appear radical, but they are actually very simple, removing fragmentation of control and responsibility, and intended to achieve one primary goal – to ensure that building design and construction is performed as it should be.

If these proposals are not adopted, I am certain that not only will someone be able to write the same sort of damning ‘open letter’ in 5 or 10 years time, but the situation will be even worse, and positive change will be even harder to implement.

2. Rationale and Recommendations for the Proposed Framework of Reform

(a) At present, accepted practice in the structural engineering profession and the construction industry in NZ are driven by the worst practitioners, not the best, and the resulting buildings in general do not comply with the intent or spirit of construction legislation, good practice, or the public’s wishes.

(b) If the majority of people in the profession, the industry, and the regulatory authorities had the right attitude, and were prepared to speak out, and back up their words with actions, the construction sector would be functioning very well indeed, with no need for reform.

Unfortunately, this is not the case, and the ‘leaky building’ crisis is just the tip of the iceberg of problems in the entire industry.

(c) If the ‘open letter’ had been written a few years ago, it would have been ignored, yet every example of poor practice described in the letter has been common for years.

Ironically, the ‘leaky building’ crisis, recent fatal collapses on building sites, the widespread failure of many development and construction companies, with the flow on effect on otherwise sound companies, the insurance industry shakeup after the
destruction of the World Trade Center, growing public disquiet, and embarrassment for politicians, have created an environment where not only has the letter been considered on its merits, but where comprehensive reform can be achieved quickly.

(d) In NZ, and in other English speaking developed countries, the last two decades have seen a paradoxical obsession with overpriced real property, at the same time as an obsession with ‘minimum’ cost has led to a severe downgrading of the building stock, its long term true value and usefulness, and its safety, particularly relating to extreme events, such as earthquake and fire.

This decline in quality occurred just as research and experience was leading to a correction of previous practices which could not handle extreme events, such as the minor gas explosion in the Ronan Point highrise flats in Britain, which caused one corner of a 16 storey apartment building to collapse.

(e) In Europe, and most of the north-eastern parts of the USA, where modern, Western style cities and economies developed, if buildings could withstand snow loading, they would perform adequately from a structural point of view, and the greatest civil danger to buildings was from fire or floods.

In the more recently developed areas of the USA, New Zealand and the like (for example, Los Angeles has been a major city for little over a century), seismic loads, volcanic eruptions, normal wind loads, hurricanes and tornadoes produce loads that require far better structural performance of buildings than has historically been the case.

This requirement for better structural performance is compounded by ever increasing, arbitrary architectural demands, the economic requirements to use materials of much higher strengths than previously, and the impact that widespread structural damage has on the communications networks and the economy as a whole.

In these new areas, our research and recording of the magnitude of extreme events is often found wanting. For example, in the North Sea, which has been traversed by thousands of ships for centuries, and been the subject of scientific research for many decades, the first offshore oil drilling platform was subjected to its 100 year return period design storm five times in the first year. Similarly, in the Northridge earthquake, buildings experienced seismic loads up to 2 to 3 times the code design levels, which were already considered to be based on sound research.

A sensible, conservative approach is necessary for the design of buildings in these areas, and new knowledge must be included into standard practice without delay.

(f) There has been mounting evidence over the last decade (through observed performance and some isolated, fragmented but long overdue testing) that precast floor systems as used in NZ may perform very badly in earthquakes. What have the standards writers or engineers done about it – virtually nothing, and at this rate, nothing will be done.

(g) The multitude of governing bodies, standards committees, professions, practitioners, developers, project managers, contractors and sub-contractors (many of very temporary nature), and the present laws and their implementation, have created a confused mess where far too many people can avoid their basic responsibilities.
The fragmentation of control in the industry has lead to terrible inertia, with very poor and slow lines of communication. Solutions to problems, when finally produced, tend to be incomplete and ambiguous, leading to even further inertia and problems.

It is difficult if not impossible to correct structural and other building defects once constructed. The only people who benefit from the present environment are the worst of developers and builders, and lawyers. Unfortunately, most people who have committed their money to buying a house or apartment do not have the large sums necessary for legal fees and remedial works should things go wrong.

Things shouldn’t go wrong in the first place – this is a recent occurrence in NZ.

The Territorial Authorities have proven incapable of performing their role under The Building Act. Two of the main reasons have been the meddling of local body politicians who do not understand what they are dealing with, and a pervasive management culture that attempts to measure performance by such ridiculous standards as to how quickly building inspections are performed, not how well they are done.

Useless rhetoric pervades debate, without regard to the actual reality of things. For example, much is made of performance based building control over prescriptive based building control.

The two have really always gone hand in hand.

The old prescriptive standards formed a useful, everyday guide for the construction industry. Alternative methods could usually be followed, provided they were adequately vindicated by test or similar.

One of the main problems with the NZ Building Code, as described by an architect involved in its development, was that it was supposed to set a minimum standard, aimed at raising NZ’s already poor building standards, not the negotiable maximum standard perceived by most developers.

With the vast range of building forms now employed, continuous developments in seismic engineering, and the removal of such stabilizing influences as the Ministry of Works and Development, the long established system of controlling building design and construction through national standards has been shown to be sadly wanting.

Advertisements, which have found their way into NZS 3604 and NZS 4229, have no place in such standards).

Traditional national standards are fine for the manufacture of materials or the design/testing of a very specific type of product, such as electric stoves or petrol motor mowers, but they are proving to be very poor for the design aspects of the NZ construction sector, particularly with regard to rapid response to change.

I believe the major structural ‘standards’ should be written and maintained by the Structural Engineering Authority that I propose to be formed under the BIA. This is
similar to the practice in the USA, where the AISC and ACI produce the most widely used structural steel and reinforced concrete standards.

(m) A major cause of the decline in the quality of building work is the fact that the initial ‘owner’ driving construction is usually not the long term owner/occupier. The case is usually one of developer on selling to occupier or ‘investor’. This is even the case where apparently sound property investment or insurance-type companies appear to be both the developer and long term owner.

One arm of the investment/insurance company acts as the developer, seeking to cut short term costs, irrespective of the impact on life time cost. The building is then ‘sold’ at the maximum profit to another arm of the company, which then sells units in the property to individuals, who pay for the initial shortcuts through increased operating costs, high maintenance and early building replacement.

(n) The ‘market mechanisms’ so much in favour since 1984 have not even addressed the most ‘market oriented’ problems in the industry.

Structural engineering certainly appears to be what economists call an ‘irrational industry’, as evidenced by the constant cutting of fees, even in times of very high and increasing demand for structural engineering services.

Despite falling fees, the personal incomes of structural engineers and draftsmen have generally held up well, but this is because the amount and quality of work done on each job has declined drastically.

Similarly, large building contractors have slashed their margins for risk and profit from about 25% thirty years ago, to about 6% today. Look at the failures that have occurred as a result.

The desperate shortages of skilled labour should have been corrected automatically by market mechanisms, but it hasn’t.

(o) The devastating boom and bust cycles of the construction industry have to be controlled by some direct government mechanisms.

Real estate agents, car salesmen and advertising account executives can be had in next to no time, but it takes at least 5 years to train up a good tradesman, and 10 years to train up an experienced, stand alone engineer. In order to train the young people, you also need to hang on to the experienced older people, and give them enough time to pass on their knowledge.

(p) The manager/accountant driven ideas that the work force can be downskilled so that minimum wages can be paid is nonsensical and must be driven out of the profession and the industry by direct action and eternal vigilance.

Any competent person involved in an industry that actually has to produce something useful knows that highly skilled experienced workers are far and away the most productive and cost effective.
In the present environment, many competent engineers and tradesmen have to spend much of their time sorting out the mess of others, instead of doing something productive.

(q) For a variety of reasons, including the very poor (but completely avoidable) economic performance of NZ since 1984, many trained engineers and tradesmen have left NZ for Australia and Britain. Both of these countries, but Britain in particular, have construction sectors that are in better state than in NZ, but there is serious concern in these countries about the state of their own construction sectors.

In the accompanying photocopied sheets is an article from Peter Miller in Engineers Australia Nov. 2002. He reports from a FIDIC conference on the continuing trend in the USA to treat architectural and engineering services as commodities, with increasing design errors, and the problems with professional indemnity insurance in Australia.

Similarly, from Britain, John Allen in the New Civil Engineer Int., Jan. 2003 reports on the exodus of talent from the engineering profession, and the difficulty in recruiting new staff. “If this situation is not addressed soon, we are in danger of witnessing the death of the British construction industry in as little as 20 years time.”

(r) The NZ politicians and public as a whole have to made aware of what structural engineers actually do, and the enormous amount of skill and hard work that is necessary to do each job as it should be done. They must also be made fully aware of the regulatory environment and income levels necessary to maintain engineering and construction in a sound state.

There is actually no need to use a structural engineer for private, commercial, or industrial buildings. Provided people and companies are prepared to build on good, reasonably flat land, and use masonry and timber construction in forms covered by NZS 3604 and NZS 4229, they can save the time and money they presently waste on structural engineers.

(s) The NZ politicians and public as a whole (and unfortunately, also large sectors of the design and construction industry) have to be made to realize that structural engineering is not a commodity able to be reduced in cost at will.

Unlike mechanical and electrical engineers, who at least in manufacturing situations can spread their design costs over thousands or millions of mass produced units, nearly every structural job is unique. Each design job is further complicated by NZ’s highly variable topography and soil conditions, our relatively small land plot size in commercial areas, the highly variable existing building stock, which must often be modified or underpinned during construction, and imposed constraints that no other professional, or even other type of engineer, has to put up with.

Most people freely accept that a car will have four sensibly positioned wheels, with the same size tyre on each, and people expect road lanes to be simple and continuous, and machines are allowed straight shafts and ready access for installation, inspection, maintenance and replacement. However, rational, efficient structural layouts are difficult to achieve because of the wishes of planners, owners and architects.
I am sure that the knee jerk reaction of many people to my proposed reforms will be that they ‘do not take account of commercial reality’, that they ‘go against the whole thrust of deregulation and Government policy’, they will ‘cause delays’, and that ‘costs will increase’.

While my proposed reforms do go against ‘deregulation’, I intend this, because deregulation in the construction sector has failed abysmally. However, my proposals do take full account of commercial realities, and in the long run will lead to savings in time and massive savings in cost, particularly lifetime costs.

Irrespective of which theory is in vogue with the Treasury and the Government, gravity still acts down.

The biggest delays in the construction sector are not attributable to incompetence by the structural engineer. The highly paid ‘decision makers’ who run so many of NZ’s companies spend months and even years not making decisions, then expect construction to start immediately. Ordinary consumers are no better. I once had a secretary at work approach me at 11:40am to see if I could produce for her by 12:00pm calculations and drawings to lodge with a building consent application for a garage to replace one that had burnt down. Similarly, builders have been known to rush into architect’s offices at 8:30am, wanting new house designs completed by lunch time the same day!

During the construction phase, the amount of time that is wasted through the reissue of incomplete drawings, client changes, answering unnecessary queries, repeated correction of poor shop drawings, delays in the delivery of materials, delays on site due to poor design documentation and poor workforces, and the losses and delays that occur when contracting companies go bankrupt is absolutely staggering. Less haste, more speed.

It is an incontrovertible, well researched and tested economic fact that the better the quality of the design and the design documentation, the more economical the construction process. Economies on design and detailing are false economies. This is true for the initial cost of the building. It is doubly true for the long term cost.

The cost of real property and construction in New Zealand’s more heavily populated areas, and particularly Auckland and Wellington, are, in my opinion, far higher than they should be in a well functioning, productive economy.

The cost of existing properties and buildings has risen much faster than the income of productive workers. For example, an ordinary house in Pakuranga that cost $50,000 in the early 1980’s now sells for $250,000, but the starting salary for structural engineers has barely doubled. This unsustainable speculation has been fueled by tax distortions favouring capital gains and money flooding in from overseas chasing high interest rates.

Similarly, many building products in NZ are grossly overpriced, particularly compared with Australia.

But where have costs been cut (although not passed on to the end user)? In engineering fees, skilled tradesmen and the quality of work. These three are the foundation of a sound, cost effective construction industry.
(u) The size of the construction sector, and its importance to the economic and social well-being of NZ as a whole, are so significant that its destiny cannot be left in the hands of fly by night developers and contractors, and architects who cannot even detail a leak proof house, or speak out about the problem.

(v) Free Trade and GATS:
I will limit my comments to four items.
(i) Because of the high ‘human’ content in the supply of services, and the ‘McJobs’ effect, which recognizes the fact that most services cannot be supplied any faster than they are already, the only way that service costs can be cut is by cutting the incomes of the service firms and their employees. (Can your dentist do a good job any quicker? Mozart’s 40th symphony still takes the same time to play as it did when it was written).
(ii) Some NZ engineering firms hope to use GATS to get work in other developed countries by offering low fees based on the exchange rate and low NZ salaries. But that is exactly the same method by which consultants from Third World countries will seek to get work in NZ.
(iii) None of the senior Government and Opposition politicians I have written to about this have attempted a reply to my simple questions, except Dr Wayne Mapp, who put his faith in local standards ensuring that overseas designs would be adequate. What he easily overlooks is that my open letter shows that even NZ trained engineers are not doing at least seismic design properly, and with engineering fees at Third World levels, there soon won’t be anyone left in NZ who can ensure the foreign designs comply (and what a terrible job checking such designs would be). I also pointed out that it will not be long before NZ’s special seismic design requirements are challenged as an unnecessary restraint of trade.
(iv) The reality of ‘free’ trade, as opposed to ‘fair’ trade, are highlighted by the accompanying letter on the MMT fuel additive under NAFTA. If ‘free’ trade is so good, why does it have to be accompanied by punitive actions, particularly if you seek to withdraw from the agreements.

(w) During my interview with the SESOC Management committee, I raised my idea for the preparation and dissemination of a simplified set of design guides and requirements for basic aspects and elements of structural design and drafting, because as outlined in my letter, the only consistency seems to be in doing things the wrong way.

One of the committee members (who works for a company guilty of many of the worst practices I condemn) exclaimed “You can’t do that, because that is ‘boxes’, and when something is outside one of the boxes, no one will know how to design it!” This is nonsense, and at least if we can get everyone designing the basic elements correctly, we will have a much better chance of getting the more complex elements designed correctly.

Of particular need is a set of clear and mandatory guidelines for the design of actual buildings, and the design of major systems within buildings, for example, transfer diaphragms.

Neither the steel structural standard nor the concrete structural standard outlines the requirement for the design of an entire building. The effect of this is clearly evidenced in many of the examples in my ‘open letter’, where, for example, some elements such as
reinforced concrete columns may be designed well, but the entire building is deficient because it has not been the product of sound engineering judgement.

One of the worst examples of this type of deficiency not covered in the letter would be the seismic resistance of foundation systems for high rise buildings in the central business districts of Auckland and Wellington. Much of the supposed lateral resistance is designed to come from bending in piles around the perimeter of the site, or from walls to basement car parks bearing against the neighbouring soil. But what happens when the neighbouring site is excavated for a new building? The new construction will be designed to underpin the other building’s foundations as necessary, but it is never designed to take, nor can it reasonably resist, the lateral loads from the other building’s foundations. Serious consideration should be given to requiring such lateral loads to be resisted well within the boundaries of each property.

(x) Structural engineering must be the only profession where technical excellence and competence is not only not valued, but a positive impediment to having a financially successful and truly rewarding career. Can you imagine a first rate criminal barrister, or litigator, or tax expert being ranked near the bottom of his firm for the whole of his career. Can you imagine a leading specialist or surgeon ranked and treated as professionally inferior to a cosmetic medicine technician or an admissions clerk?

If this is not changed, and the entire environment of mismanagement, lack of training, under resourcing, irrational pressure and lack of job satisfaction replaced by a sensible, professional environment, there will be no competent design and construction engineers in a few years.

There has to be a concentrated, intensive recruitment campaign to get the most suitable school leavers to enter the structural engineering, drafting, and construction fields, and the promises of a rewarding career (which it certainly can be) have to be made good.

(y) The insurance industry must be directly involved in the reform process. If I was an underwriter, I believe I would have a right to know the likely performance of a building under events which could lead to a claim. At present, it appears that every multi-storey building built using precast floor systems is a seismic risk, yet the insurers are lead to believe that such buildings are at the leading edge of seismic resistant construction.

Although the total insurance business for NZ buildings is tiny in world terms, it is highly significant to NZ, and to individual underwriters overseas.

Irrespective of what the law may be regarding building control, if insurers don’t want to insure a type of construction because it is unsafe, they won’t, and no one can make them.

This is an extremely powerful weapon to force positive reform within the construction industry.

It also provides a mechanism through which the better structural engineers and contractors will have an advantage over the bad ones. If, through the bulletin of building failures that I describe below, insurers could assess the truly better designers and builders, they could offer lower premiums over the life of the building for such designers
and builders. This would give a direct economic reason for owners to use these firms, even though the initial cost might be slightly higher.

(z) Law reform, without specific action to ensure the intent of the law is enacted in the real world, is a waste of time. As I understand it, the Soviet Constitution was one of the finest ever written. It just wasn’t implemented.

(aa) Far too much of the ‘skulduggery’ that goes on in the construction industry is treated as a ‘civil’ matter, when in fact it is criminal. I recommend that several new criminal offences be legislated for, which I will call ‘Criminal Building Endangerment’, and ‘Criminal Building Fraud’.

These would carry mandatory minimum severe fines and imprisonment, would be actively enforced by a well staffed police bureau funded by additional construction industry taxes if need be, and would be applied to both companies and individuals involved in illegal activity. Flight overseas by offenders would be no escape; extradition would be actively pursued, with such additional cost carried by the offenders.

Examples of ‘Criminal Building Endangerment’ would include the following type of actions:

(i) The deliberate omission of reinforcing.
(ii) The covering up of grossly unsound construction, such as severely honeycombed concrete in major structural members, without effecting repairs or advising the supervising authorities.
(iii) Repeated disregard of sound design and construction practices.

Examples of ‘Criminal Building Fraud’ would include:

(i) Building houses that are knowingly faulty, selling them, then clearing off.
(ii) Building maintenance contractors who do not do the work they claim to do, e.g. the ‘Pomnie roof painters’ that featured so many times on Fair Go.
(iii) Not rectifying building faults in a reasonable time.

One terrible example of both criminal endangerment and criminal fraud was featured on Fair Go several years ago. A brother and sister, in their eighties, and originally from Austria, had bought a home in the rural South Island. The ‘builder’ had put up 2 storeys of essentially unmortared and unreinforced blockwork. The ‘foundations’ consisted of a ‘smear’ of concrete only 75mm into the ground. The only thing stopping total collapse of the building was apparently the aluminium window and door joinery. The builder moved away, and after the elderly couple had made full payment, the house was condemned as unsafe, and they had to move out.

(bb) Every person involved in the design, drafting, documentation, inspection, supervision and construction of buildings covered by the Building Act must be registered with the BIA, and hold a current practicing certificate.

Registration in this sense is not the same as the historical registration of professional engineers. The registration is to ensure that everyone (including 16 year old labourers) involved in the industry has been suitably trained/briefed, is aware of their
responsibilities, and is able to carry out their work without detriment to others, the job, and the industry as a whole.

Everyone must also carry a current practicing certificate, without which they cannot work. This will enable the BIA to bring all individuals to account for their actions.

The registration and practicing certificate provide a means for the BIA to ensure that all people in the industry receive at least the minimum of training in the form of the simple design and work guides that I propose, and a mechanism (along with mandatory re-education, fines, suspension and de-registration) for bad practice to be driven out of the industry.

An example of the sort of action that would lead to de-registration (and possibly criminal proceedings) involves cheating on the welding of large steel sections. One common criminal method of appearing to complete large welds joining thick steel sections is to fill the joint with pieces of steel rod, then seal welding over the top, so that it appears that a full strength weld has been made. This apparently happened on one large column in the new Getty Museum in California, and the joint fractured under small earthquake loading.

(cc) There is absolutely no reason to not eradicate bad practice from the construction sector once and for all.

All I am asking for is that architectural and engineering design is done in accordance with well founded, sound, established principles, that the resulting construction is in accordance with readily achievable standards and tolerances, that the whole process is open, fair, consistent and fosters true speed, economy, quality and efficiency, and that in return for everyone receiving a fair day’s pay for a fair day’s work (and risk), the end user receives what they are reasonably entitled to expect.

Building elements are far too expensive not be done properly, particularly when mortgaged.

(dd) My proposed Structural Engineering Authority must become directly involved in the recruitment of young people for training as engineers and draftsmen. It must also become directly and positively involved in the content and quality of tertiary education and the continuing education of the workforce.

The emphasis must be at all times on technical competence and common sense.

The ‘watering down’ of the degree status of structural engineers must stop. In NZ, only the universities of Auckland and Canterbury should be able to provide degree courses in structural engineering. These degree courses need to be improved to better prepare the graduate structural engineer for what awaits him/her when they first enter the workforce.

Universities also have to appreciate that professional engineering is a vocation, and the undergraduate training must include essential, non-academic content, such as drafting, and the rudiments of damp-proofing and insulation in building construction.
The technical institutes that have been restyled as universities and ‘Institutes of Technology’ should get back to concentrating on the essential drafting and trade training that the industry and NZ so desperately need. Again, technical competence and common sense are paramount.

I know of engineers who are lecturing or have recently lectured in these former technical institutes, and they are most concerned about the poor level of education, and the nonsensical practices that are being introduced. For example, being a ‘university’, ‘research’ is all important, but there are no funds to do structural testing of significance, so the writing of papers based on other sources passes for research.

(ee) The importation or sale of building products that are not specifically approved for use in NZ should be made illegal.

(ff) The Building Act should state clearly that an opinion as to the structural integrity of a building or element within a building can only be given by a suitably qualified structural engineer.

(gg) The construction industry is worth $NZ6 trillion dollars a year worldwide. It is a major part of the New Zealand economy, with a total annual turnover of over $7 billion. Virtually every other part of the economy is dependent upon it, as is the continued wealth, comfort, health and safety of all members of society (most of whom are registered voters).

Yet, despite the billions of dollars of turnover, a current BIA levy of over $3 million a year, and the large and small fortunes that are made through the construction industry and commercial and private real estate, there is an unjustifiable dearth of money to fund essential, much needed, large scale structural testing.

This deficiency must be made good immediately through the reconstituted BIA, in particular its proposed Structural Engineering Authority, with sufficient funds made available through levies on the construction industry.

Recent sparse tests on precast flooring have raised all sorts of concerns. These must be fully addressed immediately, and this research funding must be on a consistent, regular basis. (I am personally of the opinion that the use of precast flooring elements should be banned until the problems with their use are resolved on a case by case basis, if they can be).

Such testing must address real buildings, not isolated, idealized sub-elements, and negative findings must be addressed immediately through positive action by the proposed SEA. The present delays in addressing faults are unforgivable.

(hh) To inhibit irresponsible practices, and to aid accountability, a rigorous procedure regarding the ‘stamping’ of drawings at each stage of the construction process should be introduced, with each ‘stamp’ including full names, registration numbers, insurance policy numbers and legible signatures.

(ii) One shining example of what one committed, competent person can achieve in promoting innovative, sound construction is Charles Clifton of the Heavy Engineering
Research Association. The steel structures standard NZS 3404, which is largely the result of his efforts, can rightly be claimed to be a world leading structural and seismic design standard. He gives a great deal of thought to ensuring that innovative products will perform as assumed, and is widely sought after for advice and guidance, because of his high level of technical competence, and willingness to help. Compare this with the situation in the reinforced concrete industry. Charles Clifton seems more concerned than anyone about correcting the deficiencies in this competing sector.

(jj) Given the enormous value of the construction sector, in money and safety terms, the very poor consideration given to responsible people involved in writing standards and other regulatory documents has to change. Not only does the actual research and writing go unpaid, but until very recently, all air fares and other travel expenses had to be paid for by the individuals concerned.

To add insult to injury, if the document is a NZ Standard, what was created by the engineering profession, it is sold back to the profession at hundreds of dollars a copy (not to mention the numerous errata that usually have to be corrected).

(kk) In addition to the basic design and construction guides that should be developed, industry wide standard specifications for basic structural steel and reinforced concrete construction should be written to replace the myriad of usually very poor and out of date specifications written by each consultant.

These specifications would be similar to the basic American Institute of Steel Construction Code of Standard Practice for Steel Buildings and Bridges, and would greatly assist the default use of good practice. Chapters 14 to 16 of NZS 3404 already serve as the basis for a good general steel specification.

The consultant’s steel and concrete specifications would simply deal with any non-standard or job specific items not covered by the standard specifications.

(ll) Justification for the need and practicality of the approach I recommend can be found in The Construction Contracts Act 2002.

Despite all the permutations and combinations regarding claims and payments on building work that have been argued about in the courts over the years, this is a relatively simple, one dimensional problem, but such necessary legislation has been long overdue.

It could be argued that this legislation infringes on the ‘free market’ rights of adults and companies to enter into valid contracts, and some details in those contracts, but the legislation is clearly needed.

I contend that my recommended organisational and legislative reforms are just as necessary, as the matters to be dealt with are far more varied and complex.

(mm) Similar to the need for legislation to override ‘pay when/if paid’ clauses in contracts, similar legislation is necessary to tighten up certain clauses in construction contracts which are used to cover deficiencies and omissions in design documentation, at the expense of contractors, and more particularly, sub-contractors.
The contract documents for all projects above a certain small size should be required by law to be accompanied by a detailed and accurate schedule of quantities. The production of such schedules can even now be done automatically by a range of 3D CAD packages.

In all construction projects involving structural engineers, the structural engineers should be entitled to know by law the fees being paid to all of the other consultants, including project managers, and in the event of a building to be sold or leased upon completion, the fees paid to the lawyers and real estate agents concerned.

A market cannot operate without price information.

The concept of ‘building certifiers’ as included in the Building Act should be done away with. They are not the practitioners and ensurers of excellence they are supposed to be.

A bulletin of all design and construction ‘failures’ should be published at regular intervals. This bulletin should be freely available (subject to a nominal fee) to all in the construction industry, the insurance industry, and any members of the public so interested.

Failures would include all structures submitted for building consent with deficient information or structural design faults. Similarly, all construction work submitted for inspection that is deficient, and construction work that actually fails under serviceability or strength conditions, would be considered a failure.

The bulletin would include the names of all parties concerned who are at fault, and a brief summary of the problems. Sufficient diagrams would be used to clearly describe important details concerned.

Secrecy clauses included in the settlement agreements resulting from building disputes should be made illegal. It is essential that information on poor practices is made public, so that positive correction action can to taken by the industry as a whole where needed.

Motor vehicles are regularly tested by independent reviewers, concerns regarding safety are widely disseminated, and recalls for even minor faults are broadcast through the news media. It is usually far harder to correct defects in buildings than in cars. It is better not to have the defects in the first place.

The justification for this bulletin includes:

(i) A market cannot operate without information as to the quality of the goods and services being traded.
(ii) It is important for a client or an insurer to be able to assess the true track record of the designers and contractors concerned.
(iii) It would act as an incentive for ‘couldn’t care less’ practitioners to take more care.
(iv) The very frustrating and counter-productive practice of submitting incomplete documents for building consent so as to meet unreasonable contract deadlines, hoping to follow up with more complete documents when questions are asked, would be much reduced.
(v) More consideration would be given to the time allowed to complete jobs.
Design and construction ‘failures’ should lead to investigations back into previous projects the persons concerned were involved with, to ensure that these mistakes are not indicative of a pattern of unacceptable performance.

(rr) The Building Act should be modified to include specific requirements that:
(i) Given that buildings are required to remain sound for at least 50 years, a realistic allowance for the time required to design, review, construct and inspect buildings shall be made by all parties concerned.
(ii) The design process for buildings usually requires several design cycles and successive refinements, as well as communications and accommodations between various parties, and detailed coordination and checking prior to submission for building consent. All parties involved in construction must make due allowance for this fact.

(ss) Three truths that must be accepted by all in the industry, but particularly managers and project managers, are:
(i) If it’s behind programme and/or over budget, then the programme and/or budget are wrong.
(ii) If there’s not enough time or money to do the job right, then there’s certainly not enough time or money to do the job wrong.
(iii) Management is responsible for at least 90% of the problems and defects in business (W. E. Deming).

(tt) Legislation must be passed to allow for and enforce minimum scale fees for professional engineering services and site inspection services.

(uu) Full tax deductibility for employees must be reintroduced regarding at least structural software and textbooks.
1. The following are a basic assemblage of factors that I believe have interacted to lead to the decline in the profession and the construction industry. Time constraints mean that I have not been able to include specific examples and supporting evidence. The order is random, in order of neither time nor importance. Again, time constraints have prevented a more logical arrangement, and for this I apologise.

Please note that my comments about the actual structure of university education may be somewhat out of date, but my comments about the actual content are not.

2. When all is said and done, the people most responsible for the decline in the performance of the structural engineering profession are structural engineers themselves, mainly due to their inability to co-operate and act effectively to resist outside destructive influences, and the closed mind, short sighted, selfish attitude of far too many engineers.

However, as part of the most productive profession of engineering (the other professions being medicine, law, architecture, dentistry, pharmacy, surveying and accountancy – specialist vocations where university training is required and where the governing professional bodies and their members have a duty of care and responsibility not just to their clients but society as a whole, its governing institutions and all members of society), structural engineers have been subjected to a large range of external forces, often overwhelming, over which they have little control, and which have not affected the other professions to anywhere the same extent.

3. Historically, in English speaking countries, professional engineers have had a very low profile and low standing, with a large section of the public having no idea they even exist. This has been particularly true in Britain, which, despite leading the industrial revolution of 1750-1850, soon moved into what Professor Porter would describe as a Stage 4 economy, where the preservation of wealth is of more concern than creating it. Such an economy values the ownership of land, shares and money, tax avoidance, and associated fields such as banking, share broking, insurance, the law and accountancy.

Manufacturing industries tend to be long established, and the present owners and shareholders treat these industries as just a static asset from which the maximum of profit is to be extracted in the shortest possible time.

Through our long association with Britain, these attitudes are entrenched in New Zealand.

4. Very few New Zealanders (especially through the education system) have ever had to design anything of a technical nature, and simply do not appreciate the complexity involved or the time and experience required to do design well.

Their concept of design would be an Italian modelling clay to indicate the shape of a car, or a house/garden/clothing designer doing rough felt-tip sketches.

The widespread misconception that architects are responsible for the structural integrity of houses, commercial buildings and even bridges does not help.
To a large extent, many people seem to think that contractors and tradesmen just know how much structure to put in, and how to construct it.

Similarly, you will get draftsmen and contractors who will argue with engineers who seek to correct them. “I know what I’m doing, I’ve been doing it for 30 years and never been corrected yet”, they will argue, until finally they have to admit their error, when it becomes “Oh, I’m just a draftsman”, or, “I’m just a contractor who has to price to the drawings”. Can you imagine a nurse arguing in such a way with a surgeon, or a clerk to a judge, unless their was clear gross negligence on the professional’s part.

5. Very few lay people would think that they could carry out a surgical operation or stand in a courtroom and put forward a complex legal argument in a civil or criminal trial, but most people have at some stage mixed and placed some concrete, nailed a piece of 4x2 in place, or done some painting. Therefore, they don’t hesitate to criticise the ‘excessive’ structure that is required when an engineer gets involved.

6. There are no simple grand theories that govern structural engineering, such that a young genius (or thinks he is) can command the field at a young age. Because of the near infinite variety of site conditions, soil conditions, historical building practices, existing old structures and the like, years of experience are required, and little is simple through no fault of the engineer.

James Clerk-Maxwell was a Scottish scientist whose portrait hung on Albert Einstein’s wall (along with Newton and Faraday). In 1853 I think, Clerk-Maxwell sat down with pen and paper and calculated that a charged particle accelerating in a magnetic field would release waves that travel at the speed of light. This was the theoretical foundation for our first understanding of electro-magnetic radiation (infra-red, visible light, infra-red, microwave, X-Ray and gamma), and Roentgen confirmed the theory 40 years later with his work on X-Rays. But Clerk-Maxwell’s contributions to the analysis of structures was very limited, and is now taught to first year engineering students, because he simply lacked the ‘computing machines’ that are necessary to solve the equations which govern structural analysis.

Andrei Sakharov (I think I have the name right; I think he won the Nobel Prize – he is known as the ‘Father of the Soviet hydrogen bomb’) decided against being an engineer, because it was ‘too difficult’. Instead, he preferred the physics of the very large and the very small, where pure mathematics rules.

I don’t mean for one moment to compare the typical structural engineer with these two geniuses, but the sort of difficulty that Sakharov was referring to can confront a structural engineer at any moment:

A potential client comes in to the office on a Thursday about a new job you’ve never heard of before. He’s got a contractor setting up on site who’s given him a firm price to build a two storey concrete building with surface footings, based on the architects sketches, and the building’s been leased off the plans to a tenant who needs every square metre of space. Will we have the engineering drawings completed by Monday (4 days away), so that construction can start?

It will turn out that the site has very soft peat and silt deposits, overlain by thin broken lenses of basalt lava flow, which are too thin to support the building weight but will make piling very difficult. Also, on two sides, there are old, unreinforced brick buildings which must be underpinned.

Not only has the contractor not allowed for the cost of piling, but the building layout cannot accommodate the physical width of the necessary permanent underpinning of the adjacent buildings.
(I exaggerate only very slightly with this example. Not only does the engineer have to deal with the physical reality of the site that everyone else can ignore once the engineer is involved, this example indicates the lack of appreciation of what engineers do, and how much time they need to do it, even by those people who work with them all the time).

7. Unfortunately, even when engineers ‘succeed’, they are perceived to have ‘failed’. They are also almost always perceived as adding cost, never value. Consider the following two examples:

(a) A man worth several million dollars goes into his doctor feeling unwell. The GP determines something is seriously wrong, so the man is directed to a specialist, where the GP’s fears are confirmed. The man is whisked off for immediate surgery, which is successful, but requires a lengthy period of recovery. The direct cost of medical treatment is $50,000 (covered by either the public health system, medical insurance or direct cost), and the consequent costs during the period of recovery, including lost business opportunities, total $150,000. But the man is more than happy. He is more than satisfied with the GP, specialist, surgeon (and indirectly the anaesthetist, etc), and feels he has recovered the forty odd years of life he thought he would lose.

A similar man, worth millions, buys a cliff top property without engineering advice, and commissions a ‘leading’ architect to design a multi-million dollar house, where he can enjoy the fruits of his labour, and through which, his wife can “make a statement”. Eventually, the man seeks engineering advice regarding the section, and it turns out that apart from the fact that the house requires significant engineering input because of the way the architect has ‘designed’ it, stabilising work is necessary to prevent collapse of the cliff top part of the section, and a significant cut necessary to accommodate the house requires a major retaining structure. Engineering fees are $10,000, and physical works will be $90,000. The works will enable 50 years worry free use of the site, but the man and his wife are not happy, because of the ‘cost’ that the engineer has added.

(b) A well designed building is required to have a factor of safety (strength divided by load) of approximately 1.6 for gravity loading, and 1.1 for extreme wind and seismic loads. The deflection of the structure under service and ultimate loads is also limited to avoid non-structural damage, and inconvenience to the occupants of the building.

If an engineer succeeds in meeting these design criteria, he will inevitably be open to accusations of ‘over designing’. For example, when trying to make contractors rectify deficient work, the contractors will often argue that buildings have enormous factors of safety, and this will protect their deficient work from failure.

Then of course, if the building does show signs of strength or deflection distress, the engineer has obviously failed. The engineer simply cannot win.

8. Under normal circumstances, engineers can never ‘win’, they can just not make mistakes. Because everyone (rightly) expects buildings to ‘stand up’, not deflect too much, and all the pieces fit together during construction, a successful design by the engineer is taken for granted. We are the victims of our own success as a profession.

People and companies involved in litigation will pay very large premiums to lawyers to try to shift the chance of victory from 50:50 to 60:40, but because buildings are meant to stand up, why should they have to pay for it.

9. Structural engineers in particular suffer from two major disadvantages that do not affect other civil engineers as much.
One is the minutiae of detail we must often deal with, whereas civil engineers for example can deal with moving single, massive amounts of earth, and detail drainage systems that can be well detailed with simple line drawings. Similarly, geotechnical engineers can look at the overall characteristics of the site, without getting bogged down in the complex detailing of each foundation element. This is not to say their job is easier, it’s just that they can often deal with a large value of construction with a minimum of design and detailing.

The other major disadvantage is one of the main reasons for the decline in the structural engineering profession.

In the design of civil works such as dams, roads and coastal protection works, the civil engineer tends to be the single consultant to a (at least somewhat) technically competent client. (There may be planners involved, and the like, but they tend to be off to one side).

However, with structures, the engineer is often well removed from the usually technically ignorant client. There will usually be a project management company, an architectural firm and a quantity surveying firm involved, and they usually have better contact with and influence over the client. In addition to the structural engineer, there will also be a building services engineer involved.

It is easy to see that in civil works, the civil engineer can charge a 5% fee to a client who wants to limit his fees to below 8%.

However in the structural field, the structural engineer has to live with a small fraction of the 8% fee total (now often less than 1%, while the quantity surveyor, who measures the quantities of materials used, and provides costing advice, gets around 2%).

10. Of the group of consultants usually involved in building projects, it is a simple fact that structural engineers have to be amongst the last to start, but they are the consultant who has to issue the first drawings for construction. They also need precise information from the project managers and architects which is often not forthcoming.

I have personal experience of two projects where my firm had done nothing wrong, but when problems occurred, we received letters out of the blue from the project manager of one job, and the architect in the other, blaming us for things we were not responsible for. Of course, these letters had copies sent to the client at the same time, before we had a right of reply.

11. Engineers are constantly on the back foot because of the calculations they have to perform. Even for buildings with relatively simple geometry, it can take days to enter the geometry and loads into a structural analysis program, check the data, run the analysis, and print out the building deflections and member actions. This is without doing any design.

No other job in the construction industry requires such extensive data entry and analysis before even starting drawing. Similarly, no other profession requires such extensive numerical calculations before being able to produce some useful answers.

12. Whereas architects need only consider the outside shape of a building structural member, and its surface finish, a structural engineer must consider the following:

(a) The outside shape of the building structural member,

(b) The overall structural response of the whole building, and the resulting forces on each structural member.
(c) The resulting forces at every position along each member can consist of an axial force, a shear force in the direction of the major axis of the element, a shear force in the direction of the minor axis of the element, a torsion moment, a bending moment about the minor axis and a bending moment about the major axis. These forces can vary continuously along each member for each load case, and there can easily be sixteen different load cases that must be considered.

(d) The worst effects of all of these actions for all of the load cases for all of the members must be considered.

For reinforced concrete members, these actions will be resisted by not only the concrete, but dozens of individual pieces of reinforcing steel that are placed in the concrete to resist various internal tensile forces, and to provide confinement to the concrete. The anchorage, bond strength and curtailment of each piece of reinforcing steel must be considered.

For steel members, welds and stiffening plates are often required, and detailed calculations for each are required. Composite steel beams, where the slab acts in conjunction with the steel beams to resist bending through the action of studs welded to the steel and embedded in the concrete, require extensive calculations for each of various stages of construction.

(e) Efficient modern fire design can require additional load cases to be considered.

(f) Then there are the connections which have to be designed. Figs. 1 & 2 show what engineers must design in seismic resistant beam/column joints and coupled walls, and the architectural equivalent.

(g) The NZ seismic design provisions require that most structures be ‘capacity designed’. Capacity design enables a structure to be designed for reduced seismic forces, but only if the structure is detailed in such a way to enable it to deflect backwards and forwards in a ductile manner, with controlled localised damage, but no overall or localised collapse.

This is analogous with the crash resistant design of Mercedes-Benz and Volvo cars. These cars are designed to be solid under minor collisions, but under massive collisions, they are carefully designed to deform in a controlled fashion, with the solid passenger compartment able to hold together and be pushed away from the point of impact, the engine being forced back but down, to protect the passengers, and with the front or rear sections crumpling but holding together, absorbing large amounts of energy.

Models of even the more expensive cars are mass produced by the tens of thousand, all mechanically identical, and before production, numerous prototypes are tested to destruction, so most potential safety problems can be identified and dealt with.

In contrast, virtually every building structure is unique, and very few full size buildings or building models have ever been tested to destruction.

(h) Typically in Europe, a concrete building need only be designed for gravity loads.

13. In buildings and bridges, the columns, walls and piers carry the gravity loading on the structure (structure self weight, contents, traffic, etc) down to the foundations. At each floor or deck level, a combination of slabs, beams and/or trusses (and even cables and guys) span to carry the gravity loads back to the supporting columns, walls and piers. This ‘spanning’ generates bending moments in the floor/deck members. Under lateral loads, large bending moments are generated in the columns, walls and piers.
Bending moments are the ‘penalty’ you pay for having the supports offset from the line of action of the load. They are the most costly structural action to resist. They vary as the square of the span, and the associated deflections vary to the power of four of the span. As described above, each load case generates its own bending moment pattern.

In order to achieve more economic and more practical designs, under certain conditions engineers can allow localised yielding of the structure, and redistribute moments, which change the bending moment pattern for each load case.

For the capacity designed seismic load cases, additional bending moment patterns are generated.

The beam design follows from these bending moment patterns, and must include assessment of the lateral restraint of the top and bottom of the steel beams, and how the longitudinal reinforcement in reinforced concrete beams must be lapped, anchored and terminated.

Typical bending moment patterns are shown in Figs. 3 & 4. Imagine the difficulty in producing and designing from these quickly, or checking them, or checking the designs that result from them. It is easy to see why ‘short cuts’ would be attempted.

Now compare them with the simple word processor and spreadsheet modelling that accountants and lawyers can get away with.

14. Over 50 years ago, when seismic design was non-existent and architects were often able to take full responsibility for the design of a concrete structure using empirical design charts, building layouts tended to be very simple. Now that architects don’t have to worry about the structure, and most have very poor ‘trade’ skills (they don’t know how buildings are actually constructed, or know how to detail proper weatherproofing), they get obsessed with their ‘art’, and someone else can sort out the practicalities.

15. Compared with most ‘western’ countries, the level of technical competence of contracting companies working in the commercial field is very low. Precast concrete lets them throw up a structure relatively quickly, and anything they don’t want to do can be dismissed as ‘uneconomic’ or ‘impractical’. These companies tend to be dominated by quantity surveyors, not engineers. In contrast, in America or Europe, a contractor will happily construct what the engineer has designed. Cast in-situ? No problem; they have the formwork systems available to do the job competitively.

A very good experienced architect once said to me that no NZ (building) contractor has a research and development department, and this is a very important point to remember.

16. The typical Bachelor of Engineering degree for structural engineers at Auckland University consists of an intermediate year, spent mainly in the science department, and three professional years studying for a BE (Civil).

In the first professional year, the course content is of a very general nature, and many of the papers are sat by all of the students (Civil, Chemical, Mechanical, Electrical and Theoretical and Applied Mechanics).

In the latter part of first pro, and in the remaining two years, by the time general studies, surveying, fluid mechanics, hydrology, roading, contract administration, mechanics of materials, geomechanics, public health engineering, structural analysis, engineering mathematics, management, and now I think, sustainable development, there is very little time left to teach and learn structural design, which is an enormous field in itself.
Certainly, what design is taught tends to concentrate on relatively idealised examples of parts of buildings, and not the design of buildings as a whole, which is crucial for seismic design. Steel and cast in-situ reinforced concrete tend to be the materials concentrated on.

Floor diaphragms, which transmit loads to and between the lateral load resisting elements under seismic loading, never used to be mentioned. They are very poorly covered by the NZ standards, and in professional journals. Therefore, is it a surprise that they feature so badly in my ‘Open Letter’.

Therefore, the graduate structural engineer is invariably poorly prepared when he or she starts work.

17. It is sad to say that NZ structural engineers are generally very poor managers, and even worse educators.

A fresh graduate is usually sat down at a desk and expected to churn out designs, with no decent briefing.

Even in the largest consultancies, there will be precious few if any documents which state how designs for that company are to be done.

If the graduate asks questions of the more experienced engineers, the usual answers will be of the sort:

“I’ve never worried about it.”
“It’s so stiff, it can’t rotate.”
“If you check the stresses you’ll get a fright, so don’t.”
“Well, every one else is doing it, and gets it past Council.”
“We haven’t got time to mess around.”
“Don’t analyse it, just design it.”
“Don’t bother reading the code, just do this.”

Instead of being the priority of a professional consultancy, technical education, training and development are assigned the lowest resources. The very little in-house technical training I have ever received was always scheduled for lunchtime or after hours.

18. The various pressures and constraints applied by the physical laws of the universe, clients, and other sectors of the construction industry that are described in this letter are applied to real people working as structural engineers in NZ.

The gaping holes in research, the structural standards and published design guides leave so many important aspects of design, and particularly seismic design, poorly defined at best.

The poor vocational training for structural engineers at university, their poor on-the-job training and guidance, and the lack of an overriding technical authority able to check on design practices and enforce consistent best practice leads to an ‘anything goes’ attitude to design.

The completely fractured nature of the universities, the standards committees, the promoters of steel and concrete design, the local authorities (now run by ‘managers’), IPENZ, etc, mean that so many important aspects of design are ignored.

The combination of wide spread assumptions (such as the performance of precast flooring in earthquakes, ignoring torsional loading, etc) which are now being shown to be wrong, the poor standing and lack of authority of technically competent engineers in councils, consulting firms and contracting firms, ‘standard’ practice, poor training, the lack of comprehensive guidelines,
the complex behaviour of real structures, money and time constraints, low fees, and ‘grovelling’ to get jobs all combine to drive down technical standards and allow poor designs through.

As poor designs get through, the assumptions they are based on become accepted, and the baseline is lowered. Also, there is no forum where a single structural engineer can raise a valid concern, get a straight forward answer, and if the concern is valid, get immediate corrective action enforced throughout the profession and the industry.

This whole malaise, and the nasty attitude of so many of the poorer structural practitioners, can only lead to a decline in standards over time.

19. With the demise of the Ministry of Works, not only has an excellent training ground for engineers been lost, and the perfect training ground for draftsmen been lost, the profession has lost an enormously beneficial source for technical guidance and control.

Structural engineers still cling to copies of the MOW Retaining Wall Design Guide, and the MOW Pile Design Guide.

The MOW carried out much testing and innovative design work, particularly with regard to seismic design, and sponsored much post-graduate research at the universities.

Most importantly, the MOW representatives on various standards committees provided a practical presence whose sole interests were technical excellence and public welfare.

The loss of the excellent training grounds for tradesmen provided by the MOW, NZED, NZ Railways, etc, has had a devastating effect on the construction industry. And it is not just the construction industry that has suffered. The loss has been exasperated by the ‘bean counters’ who unfortunately make up the upper ranks of NZ company management, and sit on the Boards of Directors. They have cut back in-house training for more than 15 years, and we haven’t even felt the full loss yet. This loss is critical.

20. Structural design is an iterative process, yet we are supposed to run through each stage only once. Even with staged design releases, such as the commonly used ‘Preliminary Design’, ‘Developed Design’ and ‘Final Design’, structural engineers can be trapped, because even the relatively small increase in the thickness of a wall can generate a hue and cry from the architect and the prospective tenant.

Conversely, owners, prospective tenants and architects can impose what may appear relatively innocuous changes which radically change the structural behaviour of the building.

However, architectural finishes can be changed right up to the last minute, with little or no impact on the architect’s workload or liability.

21. Ideally, structural design should consist of designing the last element to be put on first, working backwards to the foundations. Unfortunately, the design process is usually carried out in reverse, with the foundations designed and issued for construction before the building above has been finalised.

22. In general, a constructed building, particularly a reinforced concrete building, is stronger than the sum of its parts under gravity loading because of certain actions which are known but very difficult if not impossible to quantify.

In steel portal frame light industrial and warehouse type buildings, the thin profiled steel cladding can act as a stiff diaphragm, interacting with the steel frames, purlins and bracing to provide a
reserve of strength and stiffness not shown by the calculations for the main structural members acting by themselves.

While an engineer welcomes any extra strength and stiffness he can get, this can lead to bad practices I believe.

Many engineers start out designing portal frame buildings, and make mistakes relating to lateral restraint, bracing and connection design which they can get away with because of the strengthening effect of the cladding.

However, when they continue to make the same mistakes on un-clad, heavier industrial support structures, the problems of poor design and overstress outlined in the Open Letter present themselves.

In concrete buildings, arching within slabs, the strengthening effect floor diaphragms have on concrete beams, friction and the tensile strength of concrete act to make the building stronger, and prevent many potential problems with precast slabs and beams.

However, the strengthening of the beams is never considered in the capacity design of frame buildings (capacity design is described in Section 12(g) above), leading to potentially catastrophic failure mechanisms under seismic loading.

[In the capacity design of reinforced concrete frames, it is usually necessary to make the columns stronger than the beams, so that the building as a whole can rock back and forth, without the collapse of any particular level. The few tests of concrete frames with floor attached that have been carried out over the last few years have indicated that (provided the hollowcore doesn’t fall off) the effect of concrete arching is such that the overstrength of the beams should be taken approximately 80% than it is as present. But there is no central authority to immediately enforce use of the higher figure!]

Under seismic loading, it is the localised breaking of friction, and tensile cracking of the concrete, which adversely affects the precast flooring and perimeter beams, leading to the sort of floor collapse observed in the Canterbury test (which incidentally, was not meant to test the flooring - the hollowcore collapse was an unexpected by-product, though long overdue from my point of view).

23. The lack of a central authority keeping up to date with all technical developments, and ensuring rapid response by the profession and the industry has been disastrous.

24. I have recently had discussions and a meeting with a retired structural engineer, Mr Esli Forrest, who edits the SESOC Journal. He and his employers were carrying out seismic design consistent with the best modern practice almost 40 years ago. His work in the precast concrete industry was both extremely practical and technically brilliant, covering everything from zero seating details to the electron microscope study of the bond between precast and in-situ concrete. His designs were first rate, as were the construction details and practices he enforced. (He said that hollowcore was introduced against his advice). Large scale testing (privately funded by the manufacturer) was carried out almost continuously, not just to prove new designs, but to check samples of what was being produced.

If his work had been carried on with, there would be very few problems confronting the precast concrete industry today (not that these problems seem to have stopped them selling their products for use in exactly the same way as before).

Apparently, B&B Precast Concrete, where Esli did this fine work, was bought out, and all of his designs and tests were ignored by the new owners in the interest of greater profits.
He did initial work with another precast flooring manufacturer, but was dismissed when undercut by a consulting engineer who charged 1/3 the price. (Subsequently, the consulting engineer’s poor designs, and very poor construction practices, lead to failures, enormous claims and large financial settlements).

25. Unfortunately, the success of the worldwide structural engineering profession in being able to study and understand mechanics and materials to such an extent that enormous buildings and bridges can be safely and efficiently designed and constructed has come back to haunt us.

Whereas in the past, local authorities and even clients demanded full scale testing to verify new construction practices or particular new structures, now they expect everything to be able to be designed on a piece of paper, and cannot see why they should have to pay for any testing. This is a retrograde step, and allows ‘cowboys’ to flourish.

26. To meet the time and fee constraints (and general bad practices) imposed on structural designers over the last twenty years, young engineers get into the practice of ‘knowing’ what the critical load case will be in each situation, and ‘knowing’ the minimum that has to be designed, and ‘knowing’ how many calculations and drawings should be produced for a particular job, and so forth.

Practice doesn’t make perfect; it makes permanent.

Unfortunately, many of the engineers who feature in the ‘Open Letter’ are ones who learnt to do their designs this way. If nothing else, they can’t treat each job on its merits.

27. The seismic design provisions in NZ have created an environment where large beams and columns are accepted. This, along with long span precast flooring, has enabled many engineers to get away with very poor designs.

If poor NZ design practices were combined with the small column/flat in-situ floor slab buildings that are so competitive in eastern North America, Europe, South Africa and Australia, a very large number of serviceability and strength deficiencies would come to light.

28. The lack of a central controlling authority that has the knowledge, power and short response time to effectively control the structural profession and the contractors has meant that over time, standards can fall and no one can do anything about it.

The profession is starved for money to do worthwhile, realistic testing. The universities’ research is driven by what individual staff members want to study, subject to finance and suitable post-graduate students.

The people who sit on various standards committees can follow their own agendas, and a majority can ride roughshod over dissenting but sound opinions. The same people can keep reappearing on committees, so that positive change is not made.

Reviewers working for Territorial Authorities can be controlled by managers who are just that, managers, with the emphasis being on ‘efficiency’, not ‘sound engineering practice’.

Discussion between interested parties has to be through committees, which may rarely meet, and have conflicting aims.

There is no single body able to keep a watching brief, and act swiftly where necessary.
29. So much of engineering, particularly structural engineering, is based on judgement, and initial, simplifying assumptions. Also, the extremely complex behaviour of real buildings can render ‘accurate’ calculations meaningless. Therefore, in many ways, things can be ‘rough as guts’ or ‘good enough’, and work satisfactorily. Therefore, ‘vigorous debate’ can be a waste of time in attempting to change unacceptable practices, and so often the worst practitioners are those that are quite nasty and vicious, and not prepared to enter into a sensible debate.

In contrast, as soon as money is involved, extreme accuracy is important. GST is 12.5% of the value of the product or service, down to the last cent, end of story. Invoices and payments are determined down to the last cent, and accurate payment and accounting is the accepted norm.

In computer programming, virtually every character on every line of code has to be correct.

Yet good structural engineers find computer programming and invoicing far less demanding than structural design and detailing.

30. Unfortunately, in recent years the majority of our immigrant structural engineers have tended to fall into two categories. The better category consists of good, experienced engineers from Britain, Eastern Europe and South Africa. The less desirable category seems to consist of very poor engineers from all over the world.

Unfortunately, the good, experienced engineers come from non-seismic backgrounds, and aren’t able to comment on the present state of seismic design and construction in NZ. In the past, these engineers tended to work for the MOW for a few years to learn good seismic design practice, but the MOW no longer exists.

The vast range of practices and detailing that I criticize in my ‘Open Letter’ are rarely if ever put into print, and promoted in California or Japan as NZ’s ‘best practice’. In Britain and South Africa, they would generally draw derision.

31. At university, engineers tended to have an unmatched camaraderie, but this seems to be replaced in employment by a selfish and mean-spirited attitude. Back in the early 1980’s, when fees were considerably higher than they are now, new graduates were paid a pittance, and given the meanest resources to do their job.

Not only has this selfish attitude of the owners of consulting firms stifled technical competence and innovation, it ended up being completely self-defeating.

Frustration, lack of promotion and poor pay rises led engineers to jump between companies (who still don’t seem to understand the true cost of trying to replace a good worker), or start up their own. But how can a new structural firm get work? In general, only by offering to do jobs for less money and in less time, and to produce a ‘cheaper’ building. This is a downward spiral that can only, no, is, ending in disaster.
Generally Depressing Anecdotes from the Last Few Weeks
Highlighting the Radical Changes of Mindset and Action Required, and Some Examples of the Sort of Industry Guide I Propose

1. ‘Prominent’ Wellington Structural Engineers Rubbishing What I Have Said

One prominent Wellington structural engineer has, without bothering to contact me to discuss where I am wrong, been dismissive and worse regarding my “Open Letter”.

When briefing an Association of Consulting Engineers of New Zealand (ACENZ) breakfast, where even I would expect little support because I criticize these people for having let the present situation develop, this engineer described my Open Letter, but dismissed it.

Apparently one member replied “I’ve read it, and I agree with it.” Another said “I haven’t read it, but from what I’ve heard this morning, I agree with it.” Yet another, “If you think things are bad in structural, you should see mechanical engineering.” A fourth, “Maybe this is an opportunity to advertise, and increase our fees.”

I asked the Structural Engineers Society (SESOC) if I could send an e-mail to my fellow SESOC members to clarify three comments attributed to me by the various news media, that I did not make. A member of the management committee, another leading Wellington structural engineer, objected, saying that I had been acting irresponsibly. I think my actions have been the most responsible by a structural engineer in NZ for a very long time. Again, this engineer has not bothered to counter specifically anything I have written.

Dr Pet, who I had never heard of before, was brought onto the Morning Report radio programme to counter my assertions on the design and construction of seismic resistant buildings, having only skimmed over my Open Letter moments before.

He did not address my concerns at all. All he did was to repeat (with bluster) the philosophy of NZ seismic design, and its self-proclaimed world leadership. I do not attack the NZ seismic design philosophy in my open letter (although I do suggest some modifications to limit damage) - what I contend is that the design and construction philosophy is not being met. No one has come forward to say specifically where I am wrong.

During the Morning Report programme, when I mentioned as an example very bad practice relating to the construction of the top of piles, which are usually very highly stressed, particularly under seismic loading, he said I was talking rubbish, and that piles were very lightly loaded under earthquake loading.

If I am guilty of scaremongering, and what I have put in my report is such preposterous rubbish, why is it that Dr Pet has not bothered to contact me, to point out specifically where I am wrong?

With regard to my ‘rubbish’ about the top of piles, consider the following attached pictures and extracts:

(a) Fig. A shows a bottom storey column of the Olive View Hospital, that was severely damaged during the 1971 San Fernando earthquake in California. The hospital had to be demolished, but this particular column, although badly damaged, is still able to carry load because of the very good confinement provided by the substantial,
closely spaced, well anchored steel spiral around the longitudinal reinforcing. (If this spiral had been anchored as I commonly see supplied for Auckland buildings, the spiral would have burst open, and the column would have disintegrated).

A pile is exactly this same type of column, only constructed in the ground, where it must not only carry vertical load down to soil layers that can resist them, but must usually be able to carry lateral wind and seismic loads from the building down into the soil, just like a large nail can carry a lateral load into wood.

Who is speaking rubbish may be judged from (b) & (c) below.

(b) Extract B shows Sections 15.4.3 and 15.4.4 of NZS 3101:1995, and the specific requirements for longitudinal and heavy confining reinforcement in piles under seismic loading.

(c) Extract C-1 to C-3 are pages from ‘Examples of Concrete Structural Design to New Zealand Standard 3101’, produced by the Cement & Concrete Association of NZ and the New Zealand Concrete Society. This example clearly shows large bending moments and shear forces on the piles, and the consequent heavy reinforcing required.

The failure of the tops of piles in earthquakes is a commonly observed, well established form of failure.

2. Potential Collapse of House, but the Local Authority Couldn’t Care Less

On the Monday immediately after news of my ‘Open Letter’ broke on the TV, radio and newspapers, I received a phone call from a very competent structural engineer that I know.

On the Saturday just passed, he had been called around to his friends’ house. The concrete block framed house was about three metres away from a boundary, against which the neighbour’s contractor was attempting to construct a pole retaining wall in an extremely dangerous fashion.

The wall had been designed by a structural engineer, with advice from a geotechnical engineer, neither of whom had ever visited the site. The area is known to contain many weak soils. The design should have had alternate 6.5m long poles concreted 3.5m into 6.5m deep holes bored from the original ground level. Once all poles had been installed, and the concrete had hardened, the 3m excavation could have been carried out in front of the poles. Instead, the design called for a vertical cut 3m deep on the boundary, then the boring of 900 diameter holes (at 1200 centres) down an additional 3.5m. Then the poles were to be concreted in.

The design and method of construction used would have effectively created a vertical cut 6.5m deep only 3m from the friend’s masonry house. The competent engineer found the soils to be so weak that even the initial 3m cut was very dangerous. Complete collapse of the bank, and potentially fatal collapse of the friends’ house, was a real possibility.

The competent engineer got the design engineers and the local authority building inspector around, in an attempt to make them realise the danger and stop work. The building inspector said that he had better things to do on a Saturday than to be there, and believed it was no business of the local authority.
When the competent engineer complained later to the building inspector’s boss, I believe the boss said he wouldn’t have even bothered to show up.

The competent engineer was able to make the other engineers realise the danger, but the contractor said his company built several walls a week in exactly this same way.

The friends’ lawyer had to take out a court injunction to stop all work (and presumably effect temporary propping of the cut) until a safe way of construction was devised.

Even the leaky building crisis and the dog attack on the small girl prompted at least the appearance of concern from the local authorities, but not concern over building safety!

3. **Enclosed Examples of Proposed Mandatory Construction Guides**

In some of my writing, I have proposed the preparation and wide spread use of simple, comprehensive guides to improve the quality of construction in NZ. Enclosed are two extracts from publications of the sort I propose.

One is from a BRANZ House Building Guide, which I think very few builders have ever bought.

The other is from a comprehensive Irish House Building Manual, which covers a vast range of aspects of house construction, not only showing the good, but explicitly condemning the bad.
A. Indicative Answers to Specific Questions in Discussion Document

1. Extension of warranties to developers
   (a) Yes
   (b) Very few.
   (c) The sale of all dwelling units where the dwelling units are less than 10 years old from the time of issue of Code Compliance Certificates, or where modifications requiring a Building Consent have been done in the last 10 years.
   (d) None they shouldn’t already be carrying.
   (e) Yes. In the same way that a LMVD cannot sell an unlicensed/unwarranted car, the major part of the purchase price must not be paid until the BCCC has been issued.
   (f) The amount of deposit paid that would be fair and reasonable, and that this money should be held in trust so that if the unit does not get a BCCC in reasonable time, a full refund can be made to the prospective purchaser.
   (g) Yes.

2. Dispute Resolution
   (a) Delays. Cost. Massive stress. Lack of even simple technical knowledge by adjudicators in the Disputes Tribunal. Parties found liable can drag out repairs, if they are capable of doing them, and plead poverty, while still carrying on other work. A technical ‘expert’ can always be found to justify the most appalling work.
   (b) Yes, mainly due to the amount of money involved and the resultant stress. It is far easier to get out of a bad marriage than to get rid of a bad house for most people, unfortunately.
   (c) Probably a continuation of English common law, from a time when ‘property owners’ were the rich people, who could always afford legal action, and ordinary people didn’t matter. Also, there is no disincentive to prevent the developer/contractor dragging things out.
   (d) Used to be very rare, but becoming more and more common, for no valid reason.
   (e) Massive, in monetary terms but worse, in stress and resulting illness. There are thousands of cases of gross injustice, due to what amounts to fraud on a massive scale.
   (f) Up to 5% of the value of the property, provided justice is done, and if developers/contractors are liable, they pay all final costs.
   (g) Ensuring work is done right first time, that incompetent/dishonest developers/builders/etc are driven out of the industry, a Building Disputes Tribunal, registration of all participants in the building industry, and the strict enforcement of ‘no corrective action/no permission to carry on working’. Strict registration must be backed up by two criminal offences - ‘Criminal Building Fraud’ and ‘Criminal Building Endangerment’, and all moneys made from building activities must be available for redress, including all monies ferreted out of limited liability companies and into trusts.
   (h) Absolutely - who else can do it?
3. **Insurance and Bonds**

(a) The only way to encourage the voluntary supply of home warranty insurance is to have a widespread, massive increase in quality, and to have all of the fraudulent and couldn’t-care-less attitudes driven out of the industry (see below on fraud).

4. **Licensing of “Liable Persons”**

(a) Yes.
(b) Previous ‘best character’ by the individuals involved, particularly in the building industry.
(c) Wouldn’t make any difference.
(d) Shouldn’t be any extra.
(e) Enormous. If properly enforced, would help drive the unsavoury elements out of development.
(f) No.

5. **Overall costs and benefits**

(a) Yes.
(b) Yes.

B. **General Comments**

1. **Written Warranties are Already Quite Common in Construction**
Written warranties from roofing manufacturers and roofers, glazing manufacturers and glaziers, and similar sub-trades are already quite common. These are usually stated as being to the developer or general contractor (with a stated warranty period, and conditions such as regular maintenance). Each time the building is sold, these warranties should be transferred to the new owner. All the present owner has to do is write an accompanying letter to the new owner, stating that they are transferring the warranties, although it is good practice for the new owner to advise the sub-contractors concerned about the transfer.

The extension of this practice seems simple enough.

However, one major problem with the transfer of warranties (and so many other aspects of real estate transactions) is that the conveyancy lawyers so rarely advise their clients properly about these matters before contracts are entered into.

It would be good if the legislation covering building warranties made it an obligation on conveyancy lawyers to fully brief their clients about building warranties before anything is signed, with the lawyers liable for damages if they don’t.

Similarly, real estate agent should be required to check that all warranties are in order.
2. **Warranties Must Be Explicit, Not Implicit**
I have not had time to study the Building Bill yet, but all warranties must be explicit, not implicit.

They must be in simple, clear language, and must follow accepted guidelines, with certain basic requirements that cannot be excluded.

3. **Period of Warranties**
The period of 10 years is reasonable, except for extreme events such as wind loading, seismic loading, and any other extreme events that the building is purportedly designed to withstand. Wind and seismic loads are based on very long return periods, and a large amount of sub-standard design and construction passes as acceptable because it has not yet been subject to these loads. For these extreme events, the warranty period should be the appropriate return period!

4. **Present Abrogation of Contractual Obligations**
All of the recent public discussion of problems with leaky houses and declining skill standards has highlighted how the major parties involved in the construction of dwelling units are often not only unwilling to unreservedly accept responsibility for their product, they seem to have a mindset that they are the least responsible; victims of the process even.

Under contract law and common law (for the majority of new home sales), the developer is the party fully responsible for the quality of the dwelling, just as the purchaser is the party that pays the full price.

This is an indisputable fact, irrespective of who may have actually caused any defects.

Similarly, if a developer or owner/developer engages a project manager or general contractor to carry out the building works, that project manager or general contractor is fully responsible to the developer or owner/developer. All sub-contractors are the responsibility of the project manager or general contractor, with the exception of nominated sub-contractors.

Yet, despite usually being the party that makes the most money, the developers usually carry on as if they are innocent victims of events beyond their control - “We are just facilitators……”

5. **“We are just facilitators…..”**
At present, far too many developers deliberately engage the ‘worst’ architects, engineers and contractors, who will do practically anything the developers want, producing dwelling units and neighbourhoods that a poorly planned, ugly and poorly constructed. Yet the developers take their money and run. It is the people who buy these dwelling units, and the public as a whole, who suffer.
6. Widespread Massive Fraud Throughout the Industry at Present

At present, there is, I believe, massive fraud throughout the construction industry, which amounts to many millions of dollars each year.

Two of the worst examples of this fraud regarding dwellings have appeared on Fair Go over the last few years.

In the first, an Austrian born brother and sister, who were in their 80’s, bought a new house in a scenic rural part of the south of the South Island. A ‘builder/developer’ had constructed the house, then moved out of the area after the sale.

The house was supposed to be a two storey reinforced masonry structure. The ‘foundations’ consisted of a thin smear of concrete which went 75mm into the ground. The masonry was unreinforced, unfilled, and so poorly constructed that the only things stopping collapse were the aluminium window frames. The ‘builder’ who built that ‘house’ should have been sent to prison.

In the second, a ‘builder’ had thrown up twelve houses, flogged them off, then took off to Australia. At the very least, there was no drainage or damp proofing behind the masonry foundations. Concrete blocks are like a sponge to water, and the houses were a sodden mess, a disaster for their owners.

Similarly, developers, builders, engineers and architects purport that their designs/constructions comply with the NZ Building Code and relevant standards, and good practice, and sign legal documents to this effect, when in many cases this is clearly not true.

Many Territorial Authorities are guilty of fraud on a massive scale. Their ‘building inspectors’ charge fees for inspections that are totally useless and incompetent. This is fraud, and the total fees charged over the years makes this serious fraud many times over.

The fraud that pervades the construction industry in so many ways must be treated as criminal. The amounts of money involved are enormous.

To back up the proposed registration/licensing process, two criminal offences need to be covered by legislation, and rigorously enforced.

These are Criminal Building Fraud, and Criminal Building Endangerment.

‘Criminal building fraud’ is self-evident.

‘Criminal building endangerment’ covers deliberate ‘cheating’ during construction, such as the following:

(i) In lieu of full penetration butt welds to thick steel members, putting in filler plates and seal welding over the surface.
(ii) Patching over unvibrated concrete instead of repairing it properly.
(iii) Not placing reinforcing and/or grout in masonry walls.
(iv) Tack welding bolt offcuts to baseplates and endplates to make it appear that the anchor bolts are in place.

There is absolutely no valid, legal or moral reason for fraudulent or criminal activities in the construction industry. The guiding principle must be “a fair day’s pay for a fair day’s work”.

Page 4
7. Dispute Resolution Process
The problems that a friend of mine experienced with the Disputes Tribunal regarding defective work on his house were:

(i) Despite the fact that the actual technical problem (uncompacted concrete) was extremely simple, was explained in detail, with samples, and the contractor’s work was condemned by the TA and three other civil/structural engineers, the lawyers who acted as adjudicators just could not understand the technical problem.

(ii) The contractor dragged the matter out over 7 ‘hearings’.

(iii) When approximately $6,000 was awarded against the contractor, he transferred $22,000 out of his bank account, pleaded poverty, and got off with paying the $6,000 at $30 a week.

(iv) At the end of it, the contractor would still not acknowledge the extremely unsafe result of his work.

For disputes relating to houses/apartments and similar dwelling units, irrespective of total value, the first ‘court’ involved should be a specialist Building Disputes Tribunal, whose decisions are binding, but can be appealed to the court system.

This Building Disputes Tribunal could be part funded by the Building Levy, which I will propose to the Select Committee to be 1% on all construction.

However, the emphasis must be the rapid resolution of disputes, and getting problems fixed. The registration process outlined in the Building Bill, and the warranty licensing process outlined in the discussion document, must be used to enforce prompt attendance at meeting to resolve matters, and the prompt commencement of remedial works by liable developers/contractors. “No fix, no work on other jobs until fix.”

8. Fourth and Fifth Paragraphs, Page 8 of MED Discussion Document
The description of the Victorian Building Commission looks very interesting, particularly the linkage of disputes with assessing a builder’s license and/or auditing a local authority.

9. Licensing of Developers, etc.
The licensing of ‘developers’ should cover two classes at least:

(i) ‘Professional Developers’,
(ii) Homeowners who split a section, have a house built on the back, then sell one or both houses.

This licensing process should be tied as much as to the individuals concerned as well as the companies concerned, and can be used as a means of defeating the cheat of ‘phantom’ companies, that will not honour their moral obligations, and of weeding out people who should not be in the industry. ‘Silent partners’ must be illegal.

As far as Limited Liability companies and the like are concerned, just as they cannot be used as a cover for ‘conventional’ business fraud, they should not be able to be used as a cover for building fraud. Any money made through development/construction must be recoverable to make good defects.
However, people who have alterations done to their house that require a Building Consent must also be required to issue a warranty on the alterations based on the time remaining to 10 years from the date of issuance of Building Code Compliance Certificate.

10. Why Not Warranties for All Buildings
What is so special about dwelling units? Why not require all buildings to have a written warranty?

11. Cost? What Cost?
Many of the reports and discussion documents have made mention of ‘the extra cost’ of the various proposed building reforms aimed at improving building standards.

Did prices drop as standards declined? Was timber noticeably cheaper when H1 treatment was dropped? What about the present speculative bubble?

The only costs are in not doing things right, and in not strictly enforcing the regulations.

12. Punishing the Guilty for a Change
I can see no reason why any competent developer/builder/building practitioner could be opposed to a comprehensive licensing/warranty system.

Every person/company makes some mistakes – that is not the issue.

The issues are:

(i) Creating a level playing field, where the best practitioners can flourish, not the worst.
(ii) Doing things right the first time (there is no reason why good construction cannot be achieved in every instance).
(iii) If there are problems, fix them immediately.
(iv) Build genuine confidence through assured good practice.
(v) If things aren’t in writing, they’re awfully difficult to sort out.

Yours faithfully,

John Scarry, BE (Hons), ME, MIPENZ, Reg. Eng.
Appendix 2

New Zealand Building Code
Clause B1 - Structure
Clause B1—Structure

<table>
<thead>
<tr>
<th>Provisions</th>
<th>Limits on application</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Objective</strong></td>
<td></td>
</tr>
<tr>
<td><strong>B1.1</strong> The objective of this provision is to:</td>
<td></td>
</tr>
<tr>
<td>(a) safeguard people from injury caused by structural failure,</td>
<td></td>
</tr>
<tr>
<td>(b) safeguard people from loss of amenity caused by structural behaviour, and</td>
<td></td>
</tr>
<tr>
<td>(c) protect other property from physical damage caused by structural failure.</td>
<td></td>
</tr>
<tr>
<td><strong>Functional requirement</strong></td>
<td></td>
</tr>
<tr>
<td><strong>B1.2</strong> Buildings, building elements and sitework shall withstand the combination of loads that they are likely to experience during construction or alteration and throughout their lives.</td>
<td></td>
</tr>
<tr>
<td><strong>Performance</strong></td>
<td></td>
</tr>
<tr>
<td><strong>B1.3.1</strong> Buildings, building elements and sitework shall have a low probability of rupturing, becoming unstable, losing equilibrium, or collapsing during construction or alteration and throughout their lives.</td>
<td></td>
</tr>
</tbody>
</table>
Provisions

B1.3.2 Buildings, building elements and sitework shall have a low probability of causing loss of amenity through undue deformation, vibratory response, degradation, or other physical characteristics throughout their lives, or during construction or alteration when the building is in use.

B1.3.3 Account shall be taken of all physical conditions likely to affect the stability of buildings, building elements and sitework, including:

(a) self-weight,
(b) imposed gravity loads arising from use,
(c) temperature,
(d) earth pressure,
(e) water and other liquids,
(f) earthquake,
(g) snow,
(h) wind,
(i) fire,
(j) impact,
(k) explosion,
(l) reversing or fluctuating effects,
(m) differential movement,
(n) vegetation,
(o) adverse effects due to insufficient separation from other buildings,
(p) influence of equipment, services, non-structural elements and contents,
(q) time dependent effects including creep and shrinkage, and
(r) removal of support.

Limits on application
## Provisions

### B1.3.4 Due allowance shall be made for:

(a) the consequences of failure,

(b) the intended use of the building,

(c) effects of uncertainties resulting from construction activities, or the sequence in which construction activities occur,

(d) variation in the properties of materials and the characteristics of the site, and

(e) accuracy limitations inherent in the methods used to predict the stability of buildings.

### B1.3.5 The demolition of buildings shall be carried out in a way that avoids the likelihood of premature collapse.

### B1.3.6 Sitework, where necessary, shall be carried out to:

(a) provide stability for construction on the site, and

(b) avoid the likelihood of damage to other property.

### B1.3.7 Any sitework and associated supports shall take account of the effects of:

(a) changes in ground water level,

(b) water, weather and vegetation, and

(c) ground loss and slumping.

## Limits on application
Appendix 3

AS 1250 - 1981
AUSTRALIAN STANDARD

THE USE OF STEEL IN STRUCTURES

known as the

SAA STEEL STRUCTURES CODE

AS 1250–1981

PUBLISHED BY THE STANDARDS ASSOCIATION OF AUSTRALIA
STANDARDS HOUSE, 80 ARTHUR ST, NORTH SYDNEY, N.S.W.
PREFACE

This edition of this standard was prepared by the Association's Committee on Steel Structures to supersede the 1975 edition. This edition is being issued prior to the preparation of the standard in 'limit state' format in order to simplify the use of the standard and to improve clarity. The opportunity has been taken to present the standard in A4 size, thus bringing it into line with the other major structural codes. There have been no changes of major technical significance in this edition, but some editorial updating has been carried out.

The following have been amended (other than editorially):

- Clause 1.5.3 Technical Definitions
- Clause 1.6 Notation
- Clause 2.1.4 Undetected Steel
- Clause 2.2.2 Rivets
- Clause 3.1 Loads
- Clause 4.3 Plate Thickness
- Clause 4.5 Sectional Areas of Bolts, Screwed Tension Rods and Rivets
- Clause 4.5.1 Bolts and Screwed Tension Rods
- Clause 5.4.1 Equal-flange I-beams or Channels
- Clause 5.5 Elastic Critical Stress
- Clause 5.10.2 Average Shear Stress in Rolled I-beams and Channels, Plate Girders, Box-sections, Rectangular and Circular Hollow Sections
- Clause 5.11.1 Maximum Permissible Stress
- Clause 5.13.1 Minimal Thickness
- Clause 5.13.3 Vertical Stiffeners
- Clause 6.9 Bearing Stresses
- Clause 7.5.3 Pin Connections
- Section 8 Combined Stresses
- Section 9 Design of Connections
- Clause 10.1 Plastic Design—General
- Clause 11.2.3 Length
- Clause 11.3.2 Cutting
- Clause 11.4.2 Setting Out Tolerances
- Appendix B Fatigue
- Appendix C Minimum Yield Stresses for Steel to AS 1163, AS 1204 and AS 1200
- Appendix D List of References on the Elastic Flexural Torisonal Buckling of Steel Beams

Paragraph EI.5 Application
Paragraph E22 Effective Length of Struts in Triangulated Frames

Attention is drawn to the following Australian and British standards and other documents which may be required for use in connection with this standard:

AS 1112 ISO Metric Hexagon Nuts, Including Thin Nuts, Slotted Nuts and Castle Nuts
AS 1131 Dimensions of Hot-rolled Structural Steel Sections
AS 1163 Welded and Seamless Steel Hollow Sections for General Structural Purposes (Metric Units)
AS 1170 SAA Loading Code Part 1—Wind and Live Loads Part 2—Wind forces
AS 1204 Structural Steels—Ordinary Weldable Grades
AS 1205 Structural Steels — Weather-resistant Weldable Grades
AS 1227 General Requirements for the Supply of Hot-rolled Steel Plates, Sections, Piling and Bars for Structural Purposes
AS 1252 General Grade High-strength Steel Bolts with Associated Nuts and Washers for Structural Engineering (ISO Metric Series)
AS 1275 Metric Screw Threads for Fasteners (Based on ISO Recommendations)
AS 1302 Steel Reinforcing Bars for Concrete
AS 1303 Hot-drawn Steel Reinforcing Wire for Concrete (Metric Units)
AS 1391 Methods for Tensile Testing of Metals
AS 1418 SAA Crane Code
AS 1480 SAA Concrete Structures Code
AS 1531 SAA High-strength Structural Bolting Code
AS 1558 SAA Cold-formed Steel Structures Code
AS 1559 SAA Structural Welding Code
AS 1755 SAA Lift Code
AS 2074 Steel Castings for General Engineering Purposes
AS 2121 SAA Earthquake Code
AS 2214 SAA Structural Welding Supervisors’ Certification Code
AS 2312 Guide to the Protection of Iron and Steel Against Exterior Atmospheric Corrosion
AS Z5 Glossary of Metal Welding Terms and Definitions
SAA MA1—Manual on Steel Structures
SAA MA1.5—Protection of Steel from Corrosion
SAA MA1.8—Fabrication
SAA MA1.9—Erection
BS 5135 Metal-arc Welding of Carbon and Carbon Manganese Steels

Supplement 1 (PD 3343) to BS 449, Part 1 Recommendations for Design

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## CONTENTS

### 1. SCOPE AND GENERAL

1.1 Scope ................................................. 4
1.2 Standards ............................................ 4
1.3 New Materials or Methods ......................... 4
1.4 Design and Supervision ............................... 4
1.5 Definitions ........................................... 4
1.6 Notation .............................................. 5

### 2. MATERIALS

2.1 Structural Steel ....................................... 7
2.2 Fasteners and Electrodes ............................. 7
2.3 Steel Castings ........................................ 7
2.4 Concrete .............................................. 7

### 3. GENERAL DESIGN REQUIREMENTS

3.1 Loads .................................................. 8
3.2 Design Methods ....................................... 8
3.3 Other Design Considerations ....................... 9

### 4. GEOMETRICAL PROPERTIES

4.1 General ............................................... 11
4.2 Geometrical Properties for Calculating Bending Stresses .......... 11
4.3 Plain Thickness ....................................... 11
4.4 Holes ............................................... 11
4.5 Sectional Areas of Bolts, Screwed Tension Rods and Rivets ........ 12
4.6 Maximum Slenderness Ratios ....................... 12

### 5. DESIGN OF BEAMS

5.1 General ............................................... 13
5.2 Maximum Permissible Stress ....................... 13
5.3 Maximum Permissible Compressive Stress ............ 13
5.4 Maximum Permissible Stress in a Beam Bent About the Axis of Maximum Strength ........ 13
5.5 Elastic Critical Stress ............................. 16
5.6 Bending Stress for Cased Beams ................... 16
5.7 Parallels and Girts .................................. 16
5.8 Effective Span of Beams ............................. 16
5.9 Effective Length of Beams for Lateral Buckling ........... 18
5.10 Shear ............................................... 18
5.11 Bearing Stresses .................................... 20
5.12 Flange Details ...................................... 20
5.13 Web Details ........................................ 20
5.14 Separators and Diaphragms ....................... 23

### 6. DESIGN OF STRUTS

6.1 Axial Force in Unloaded Struts ..................... 25
6.2 Axial Forces in Cased Struts ....................... 25
6.3 Effective Length of Struts .......................... 24
6.4 Eccentricity for Struts ............................. 26
6.5 Splices .............................................. 26
6.6 Struts With Two or More Main Components in Contact .... 26
6.7 Struts With Two Separated Components .......... 26
6.8 Caps and Bases for Struts ......................... 29
6.9 Bearing Stresses .................................... 29

### 7. DESIGN OF TENSION MEMBERS

7.1 Axial Stresses in Tension Members .................. 30
7.2 Tension Members Subjected to Bending ............. 30
7.3 Distribution of Forces ............................. 30
7.4 Tension Members with Two or More Main Components ....... 31
7.5 Connections ......................................... 31
7.6 Bearing Stresses .................................... 34

### 8. COMBINED STRESSES

8.1 General ............................................... 33
8.2 Individual Moments and Forces .................... 33
8.3 Direct Stress Combinations ....................... 33

### 9. DESIGN OF CONNECTIONS

9.1 Minimum Design Force on Connections ............ 35
9.2 Choice of Fasteners ................................ 35
9.3 Combined Connections ............................... 35
9.4 Connection stiffeners ................................ 35
9.5 Stresses in Bolts, Screwed Tension Rods, Rivets and Pins .... 35
9.6 Design Details for Fasteners ....................... 35
9.7 Design Details for Pins ............................ 36
9.8 Welds ............................................... 36
9.9 Packing ............................................. 37

### 10. PLASTIC DESIGN

10.1 General ............................................. 38
10.2 Beams .............................................. 38
10.3 Tension Members .................................... 38
10.4 Struts ............................................. 38
10.5 Members Subjected to Combined Bending and Axial Force ....... 38
10.6 Shear .............................................. 38
10.7 Stability ............................................ 38
10.8 Minimum Thickness ................................ 39
10.9 Lateral Restraints ................................ 39
10.10 Web Stiffening ................................... 39
10.11 Load Capacities of Connections ............... 39

### 11. FABRICATION AND ERECTION

11.1 General ............................................. 40
11.2 Tolerances .......................................... 40
11.3 Fabrication Procedures ............................ 40
11.4 Erection ............................................. 41

### APPENDICES

A. Deflection ............................................ 43
B. Fatigue .............................................. 44
C. Minimum Yield Stresses for Steel to AS 1163, 1245 and 1250 .... 50
D. List of References on the Elastic Flexural-Torsional Buckling of Steel Beams .... 52
E. Effective Length of Struts ......................... 53

### INDEX

.................................................. 56

### RECORD OF AMENDMENTS

.................................................. 62
1.1 SCOPE. This standard applies to the design, fabrication, erection, repair and alteration of steelwork in structures, including foot and service bridges.

The standard does not apply to the following structures and materials:

(a) Road and railway bridges.
(b) Material less than 3 mm thick.
(c) Steel for which the value of $f_y$, used in design exceeds 450 MPa.
(d) Cold-formed members other than those complying with AS 1153.

Notes:
1. The use of cold-formed steel sections in structures is covered by AS 1153. Reference should also be made to Addendum No. 1 to BS 449 (PTL 4864), and the ASME Cold-Formed Steel Design Manual.

1.2 STANDARDS. Unless otherwise noted, a standard referred to in this standard is the current edition thereof.

1.3 NEW MATERIALS OR METHODS. This standard shall not be interpreted to prevent the use of materials or of methods of design or construction not specifically referred to herein. If it is desired to seek the opinion of the SAA Committee on Steel Structures as to whether materials other than those specified, or methods of design or construction not covered herein, are deemed to comply with the intention of this standard, details of such materials or methods, including relevant test results, shall be submitted to the Committee.

Notes:
1. It will be necessary to seek approval from the Building Authority for the use of new materials or methods.

1.4 DESIGN AND SUPERVISION.

1.4.1 Design. The design of a structure or the part of a structure to which this standard is applied shall be the responsibility of an engineer experienced in the design of such structures.

For the purposes of this standard the term 'Design Engineer' shall mean the engineer responsible for design and shall include his representative.

1.4.2 Supervision. All stages of construction of a structure or the part of a structure to which this standard is applied shall be adequately supervised to ensure that all the requirements of the design are satisfied in the completed structure. Supervision shall be the responsibility of either —
(a) the Design Engineer, or
(b) an engineer experienced in such supervision.

For the purposes of this standard, the term 'Supervising Engineer' shall mean the engineer responsible for supervision of construction and shall include his representative.

Notes:
1. Although the execution of design and supervision may be delegated to other acceptable persons who need not be qualified, Clause 1.4 requires that design and supervision be the responsibility of qualified and experienced persons.
2. The Clause does not require the Design Engineer to be responsible for supervision also unless he has been assigned this responsibility specifically. The Design Engineer and the Supervising Engineer need not be the same person.
3. Welding inspectors should be qualified to the requirements of AS 2234.

1.5 DEFINITIONS.

1.5.1 General. For the purposes of this standard, the definitions in Clauses 1.5.2 to 1.5.4 shall apply.

Notes:
Other terms having special meanings are defined in the Clause in which they occur.

1.5.2 Administrative Definitions.

1.5.2.1 Approved — according to the context, approved either by the Engineer or the Building Authority.

1.5.2.2 Building Authority — a body having statutory powers to control the design and erection of buildings or structures in the area in which the building or structure concerned is to be erected.

1.5.2.3 Contractor — the person, persons or organization agreeing under a contract to execute the work.

1.5.2.4 Engineer — a person qualified for Corporate Membership of the Institution of Engineers, Australia. See Clause 1.4.

Notes:
The definition of 'engineer' does not require that an engineer be a Corporate Member of the Institution of Engineers, Australia.

1.5.3 Technical Definitions.

1.5.3.1 Beam or girder — a structural member, other than a triangulated frame, which supports load primarily by its internal resistance to bending.

1.5.3.2 Dead load — the actual weight of all permanent construction and all permanently installed plant, equipment, and services required for functional purposes.

* American Iron and Steel Institute.
1.5.3.3 Footing — a part of the building or structure in direct contact with and transmitting load to the supporting foundation.

1.5.3.4 Foundation — the soil, sub-soil or rock, either built up or natural, upon which a structure is supported.

1.5.3.5 Gage — the transverse spacing between parallel adjacent lines of fasteners.

1.5.3.6 High strength bolt — a bolt manufactured to AS 1252 or as equivalent fastener as defined in AS 1183.

1.5.3.7 Live load — the load assumed to arise from the intended use or purpose of a structure, including distributed, concentrated, impact and inertial forces, but excluding wind, snow and earthquake forces.

1.5.3.8 Load factor — the numerical value by which the load which would cause failure of a structure is divided, to give the maximum permissible working load on the structure.

1.5.3.9 Partition — a wall employed solely for the purpose of subdividing a storey of a building into sections and which is not intended to support any load other than its own weight.

1.5.3.10 Pitch — the centre distance between individual fasteners in a line of fasteners.

1.5.3.11 Plate — a compression member including a column or stanchion.

1.5.3.12 Substructure — the general structural base upon which the remainder of a structure is supported, comprising the footings and any remaining or like wall supported by the ground.

1.5.3.13 Tensile strength — the ultimate tensile strength of a steel or electrode as defined in the appropriate Australian Standard.

1.5.3.14 Wind forces — forces on a structure resulting from wind pressure.

1.5.3.15 Yield stress — the minimum yield stress in tension specified for the grade of steel in the appropriate Australian standard.

1.5.4 Welding Terms. Welding terms shall have the meanings given in AS Z5.

1.6 NOTATION. The notation used in any of the Clauses shall have the following meanings with respect to the structure, or member or condition to which the Clause is applied, unless otherwise defined elsewhere in this standard. Unless otherwise stated, a dimension shall mean a specified dimension.

Note: The subscripts x, y denote the X-X and Y-Y axes of the section, respectively; XX denotes the major principal axis, and YY denotes the minor principal axis.

\[ A_t = \text{the gross cross-sectional area of a concrete-encased strut neglecting any caving in excess of } 75 \text{ mm from the overall dimensions of the steel section and neglecting any applied finish} \]

\[ A_e = \text{the effective cross-sectional area of a steel member} \]

\[ A_m = \text{the effective sectional area resisting shear for calculating the average shear stress or the maximum shear capacity in a member} \]

\[ B = \text{the overall width of the flange of a section} \]

\[ b = \text{the lesser clear dimension of a web panel} \]

\[ b_1 = \text{the outstand of a flange or stiffener beyond the line of connection to a web, or other line of support} \]

\[ b_2 = \text{the unsupported width of a flange element between two adjacent faces of support or between two adjacent lines of connections to other elements of the member} \]

\[ D = \text{the overall depth of a section measured parallel to the web} \]

\[ d = \text{the clear depth of the web of a rolled I-section or channel between root fillets} \]

\[ d_1 = \text{for the web of a beam without horizontal stiffeners, the clear distance between the flanges, neglecting fillets, or the clear distance between the inner toes of the flange angles, as appropriate} \]

\[ d_2 = \text{for the web of a beam with horizontal stiffeners, the clear distance between the horizontal stiffener and the tension flange, neglecting fillets, or the inner toes of the tension flange angles, as appropriate} \]

\[ d_3 = \text{the clear distance from the neutral axis of a beam to the compression flange, neglecting fillets, or the inner toes of the flange angles, as appropriate} \]

\[ E = \text{the modulus of elasticity for steel, taken as } 2 \times 10^6 \text{ MPa in this standard} \]

\[ F_{cc} = \text{the minimum compressive strength of concrete as defined in AS 1480} \]

\[ F_{cu} = \text{the maximum permissible compressive stress in an axially loaded strut not subjected to bending} \]

\[ F_{ct} = \text{the maximum permissible tensile stress in an axially loaded tension member not subjected to bending} \]

\[ F_{ctd} = \text{the maximum permissible stress due to bending in a member not subjected to axial force} \]

\[ F_{ccu} = \text{the maximum permissible compressive stress due to bending in a member not subjected to axial force} \]

\[ F_{ccu} = \text{the maximum permissible tensile stress due to bending in a member not subjected to axial force} \]

\[ F_{ctd} = \text{the maximum permissible compressive stress due to bending in a member not subjected to axial force} \]

\[ F_{ctd} = \text{the maximum permissible average stress in shear in a member} \]

\[ F_{ct} = \text{the maximum permissible stress in bearing in a member} \]

\[ F_s = \text{the yield stress pertaining to the steel used in a member} \]

\[ F_{s} = \text{the maximum permissible average stress in shear in a member} \]

\[ F_{s} = \text{the maximum permissible stress in shear in a member} \]

\[ F_{s} = \text{the yield stress pertaining to the steel used in a member; where } F_{s} \text{ is used in a dimensionally inconsistent expression in this standard, its value shall be expressed in MPa (see Appendix C)} \]

\[ f_{c} = \text{the calculated average stress in a member due to an axial compressive force} \]
$f_{ut}$ = the calculated average stress in a member due to an axial tensile force

$f_{cu}$ = the calculated compressive stress in a member due to bending about a principal axis

$f_{ts}$ = the calculated tensile stress in a member due to bending about both principal axes

$I_x = \text{the second moment of area (moment of inertia) of the cross section of a pair of web stiffeners about the centroidal line of a beam web, or of a single stiffener about the face of a beam web, as appropriate}}$

$L = \text{the span of a member}}$

$l = \text{the effective length of a beam or strut}}$

$M_{es} = \text{the calculated maximum moment capacity of a beam}}$

$M_{um} = \text{the calculated maximum moment capacity of a member subjected to bending and axial load}}$

$P = \text{an axial force, compressive or tensile, in a member or part of a member}}$

$P_{m} = \text{the calculated maximum load capacity of a strut}}$

$P_{t} = \text{the calculated maximum load capacity of a tension member}}$

$Q = \text{a force applied transversely to a member}}$

$R = \text{the reaction of a beam at a support}}$

$r = \text{the radius of gyration of a section}}$

$S = \text{the longitudinal spacing between the centres of successive vertical stiffeners in a beam}}$

$T = \text{in a flanged section, the mean thickness of the flange, or the area of the flange divided by its width, or the thickness of the flange as given in AS 1131, as appropriate}}$

$T_{1} = \text{the thickness of the flange of a section or of a plate in compression, or the aggregate thickness of plates if connected together in accordance with Section 9, as appropriate}}$

$t = \text{the thickness of the web of a section}}$

$t_{e} = \text{the thickness of the thinner of two components joined}}$

$W = \text{the total load on a beam between supports}}$
SECTION 2. MATERIALS

2.1 STRUCTURAL STEEL.

2.1.1 Australian Standards. Except as otherwise permitted in Clause 2.1.2 below, all structural steel coming within the scope of this standard shall, before fabrication, comply with the requirements of the following Australian standards, as appropriate:

AS 1131 Dimensions of Hot-rolled Structural Steel Sections
AS 1163 Welded and Seamless Steel Hollow Sections for General Structural Purposes (Metric Series) Structural Steel Hollow Sections
AS 1204 Structural Steels — Ordinary Weldable Grades
AS 1205 Structural Steels — Weather-resistant Weldable Grades

NOTE: Welded structures of steel in AS 1163 are normally satisfactory for most general construction conditions, they may be necessary in those few cases where critical parts of structures are to be subjected to thermal exposure or to severe conditions of service. These conditions cannot be defined accurately, but the type of failure should be considered possible when there is a combination of high temperature and the following factors: high strength and high rigidity, low elastic strain capacity, and the possibility of residual stresses in welding such as cracks and hot, cold, or delayed fractures. Then, it is necessary to take into account greater than 500°C temperature changes.

When the combination referred to above cannot be avoided, and adequate pre- and post-welding treatment is not possible, the use of impact-impact test in AS 1101 is recommended.

2.1.2 Other Structural Steels. If structural steels of shapes other than those referred to in Clause 2.1.1 are used they shall comply with Australian standards or with specifications issued by bodies accredited by the Building Authority, such as the British Standards Institution and the American Society for Testing and Materials. The maximum value of the yield stress $F_Y$ to be used in the application of this standard shall be 450 MPa. A steel intended to be welded shall be suitable for welding.

2.1.3 Acceptance of Steel. Certified mill test reports, or test certificates issued by the mill shall constitute sufficient evidence of compliance with material specifications referred to in this standard. At the discretion of the Design Engineer, and subject to the approval of the Building Authority, it shall be permissible to use a test certificate provided by an approved testing laboratory or an affidavit from the fabricator instead of a certified mill test report or mill test certificate.

2.1.4 Unidentified Steel. If unidentified steel is used it shall be free from surface imperfections, and shall be used only where the particular physical properties of the steel and its weldability will not adversely affect the strength and serviceability of the structure. Unless a test is made in accordance with AS 1391, the yield stress $F_Y$ of the steel for design purposes shall be taken as not exceeding 170 MPa.

2.2 FASTENERS AND ELECTRODES.

2.2.1 Steel Bolts, Nuts and Washers. Steel bolts, nuts and washers shall comply with the following standards, as appropriate:

AS 1110 ISO Metric Hexagon Precision Bolts
AS 1111 ISO Metric Hexagon Commercial Bolts
AS 1112 ISO Metric Hexagon Nuts including Thin Nuts, Studded Nuts and Castle Nuts
AS 1252 General Grade High Strength Steel Bolts with Associated Nuts and Washers for Structural Engineering (ISO Metric Series)
AS 1559 Tower Bolts with Associated Nuts and Washers (Metric Series)

Other high-strength fasteners may be used provided that they comply with the requirements for 'equivalent fasteners' as defined in AS 1511.

2.2.2 Rivets. All rivets shall comply with an approved standard.

2.2.3 Electrodes. Aluminum electrodes shall comply with the appropriate requirements of AS 1554.

2.3 STEEL CASTINGS. All steel castings shall comply with the appropriate requirements of AS 2074.

2.4 CONCRETE. Unless otherwise required by this standard, all structural and fire-protective concrete used in association with structural steel shall comply with the appropriate requirements of AS 1480.
SECTION 3. GENERAL DESIGN REQUIREMENTS

3.1 LOADS. A structure, and part of a structure, shall be designed for the loads laid down by the following standards, as appropriate:

AS 1170  SAA Loading Code
Part 1 — Dead and Live Loads
Part 2 — Wind Forces

AS 1418  SAA Crane Code

AS 1735  SAA Lift Code

Note: Attention is drawn to AS 1117, SAA Earthquake Code, where it is required to take earthquake forces into account.

3.2 DESIGN METHODS.

3.2.1 General. A structure and part of a structure shall be capable of sustaining the most adverse combination of static and dynamic forces that may reasonably be expected from all the loads specified in Clause 3.1. Except as qualified by Clause 3.2.4 or where a load factor is applicable, every part of the structure shall be so proportioned that the permissible stresses specified in this standard are not exceeded. Where applicable, a load factor used in design shall not be less than that specified in this standard.

A design method specified in this standard shall be used, at the choice of the Design Engineer, as the basis for design. The stiffness, stability and serviceability of the structure shall be such that the appropriate requirements of Clause 3.3 are satisfied.

3.2.2 Simple Design Method. The method specified herein and known as the simple design method, applies to the design of a structure, or part of a structure, in which the ends of members will not develop restraining moments adversely affecting the remainder of the structure or its parts.

(a) Rectangular framed structures, conventional method. For the purpose of this method of design for a rectangular framed structure, the following assumptions shall be made:

(i) Beams and trusses are simply supported.

(ii) All connections of beams and trusses are subjected to reaction shear forces applied at the eccentricity appropriate to the connection detail.

(iii) Members in compression are subjected to axial forces applied at the eccentricities given by applying Clause 6.4 and have the effective lengths given in Clause 6.3.

(iv) Members in tension are subjected to concentric axial forces applied over the net area of the section, as specified in Clause 7.5.

(b) Rectangular framed structures, wind connection method. If a rectangular framed structure is designed for vertical loads using (a) above, the lateral forces may be assumed to be carried by fully rigid connection action in accordance with Clause 3.2.3. Method 1, if the structure complies with the following requirements:

(i) Under lateral forces, the connections shall be designed elastically and shall satisfy Section 9.

(ii) Under vertical forces, only the welds and fasteners used in the connections shall satisfy Section 9, and the remaining elements of the cross-section shall possess sufficient deformation capacity to ensure that over stressing of the welds or fasteners cannot occur.

(iii) Where welds or fasteners are stressed by both lateral and vertical forces the combined stresses shall satisfy the requirements of Section 9.

(c) Triangulated structures. For the purpose of this method of design for a triangulated structure, axial forces shall be found by assuming that all members are pin connected. Where any eccentricity occurs, all stresses arising therefrom shall be kept within the limits of this standard.

3.2.3 Fully Rigid Design Method. The method specified herein, and known as the fully rigid design method, applies to the design of a structure, or part of a structure, in which the beam-to-truss connections have sufficient rigidity to hold the angles between the members virtually unchanged irrespective of the load.

A structure and part of a structure proportioned according to this method shall be analysed using one of the following methods:

Method 1. Adequate elastic analysis to show that the permissible stresses specified in this standard are not exceeded.

Method 2. Adequate analysis to show that the load-carrying capacity is not less than that required when a load factor of 1.05 is applied to the loads or to the load combinations in Clause 3.3.1. The structure or part shall also comply with Section 10. Nevertheless, the deflections and other movements under working loads shall not exceed the limits imposed by Clause 3.3.5.

3.2.4 Semi-Rigid Design Method. The method specified herein, and known as the semi-rigid design method, applies to the design of a structure, or part of a structure, in which the connections between the members are capable of furnishing a dependable and known degree of flexural restraint in accordance with the recommendations of Supplement No 1 (PD3343), to BS 449, or as specified below.

In cases where this method of design is employed, calculations based on general or particular experimental evidence shall be made to show that the stresses in the structure are nowhere in excess of the appropriate stresses laid down in this standard. Alternatively, where riveted or bolted connections are employed, and only used with a yield stress not exceeding 280 MPa is used, the requirements of this standard with respect to stresses shall be deemed to be satisfied if the design is made in accordance with Supplement No 1 to BS 449. The design of continuous struts shall comply with Clause 6.8.

3.2.5 Experimentally Based Design. Where a structure is of an unconventional or complex nature...
and the Design Engineer shows by full-scale or model tests that the limits on deflections, stability and serviceability, and stress or load factor imposed by Clause 3.2.1 are satisfied; the corresponding design requirements of this standard shall be deemed also to have been satisfied, subject to the following conditions:

(a) In the case of a full-scale test of a prototype structure, the prototype shall be accurately measured before testing to determine the dimensional tolerances in all relevant parts of the structure. The tolerances then specified on the drawings shall be such that all successive structures shall be in absolute conformity with the prototype.

(b) In the case where design is based on full-scale models, tests shall be representative of those to which the prototype is deemed to be subject under this standard.

(c) In the case where a structure is proposed to be constructed with due regard for the principles of dimensional similarity. The thrusts, moments and deformations under working loads shall be determined by physical tests when the loading is applied to simulate the conditions assumed in the design of the actual structure.

When it is desired to justify a structure or any of its parts by testing a prototype or model, prior notice shall be given to the Building Authority, and all the relevant tests shall be carried out to the satisfaction of the Authority.

3.3 OTHER DESIGN CONSIDERATIONS.

3.3.1 Loading Combinations. A structure or part of a structure shall be so designed that the maximum permissible stress is not exceeded under any of the following load combinations:

(a) \(X_0 + X_L\);

(b) \(0.75 (X_0 + X_L + X_d)\); or

(c) \(X_0' + X_L'\);

where

- \(X_0\) = dead load
- \(X_L\) = live load
- \(X_d\) = wind load
- \(X_0'\) = wind load causing stresses of the opposite sign to the dead load
- \(X_L'\) = that part of the dead load which cannot be removed from the structure

1.0 times the whole of the dead load tending to prevent overturning.

1.2 times the dead load, if any, tending to cause overturning, and

1.4 times the static-load equivalent of other loads tending to cause overturning.

(b) Due account shall be taken of possible variations in loading during construction and repair or other temporary conditions, so as to ensure stability at all times.

3.3.2 Lateral Forces. Adequate provision shall be made to resist the lateral forces that can occur during and after erection of a structure. Where the lateral forces cause twisting in a structural frame, provision shall be made to resist all resulting increases in horizontal shear but all resulting decreases in horizontal shear shall be neglected in the design.

When considering the effect of lateral forces, proper allowance shall be made for the strength and stiffness of all structural components as follows:

(a) Where structural components such as walls, roof, floors and other additional bracing structure are capable of transmitting all lateral forces to the substructure, it shall be deemed that the structural steelwork is not loaded by such forces.

(b) Where there are other structural components that are strong enough to transmit only part of the lateral forces to the substructure, the lateral forces shall be distributed through the structural system in accordance with the relative stiffnesses of the frame and other components, and the structural steelwork shall be designed accordingly.

(c) Where resistance to lateral forces provided by other structural components is obviously low or not ascertainable, the structural steelwork shall be designed to resist all the lateral forces and transmit them to the substructure.

3.3.4 Lateral Restraining Systems.

3.3.4.1 General. Lateral restraints required to act as intermediate lateral restraints for beams (Clause 5.9.2.2) or to reduce the effective lengths of struts shall meet the requirements of this Clause (3.3.4).

3.3.4.2 Forces. A single lateral restraint shall be designed to resist a force of 0.025\(P\) in addition to any other forces to which it may be subjected, where \(P\) is the maximum axial force in the critical flange or chord.

3.3.4.3 Stiffness. A single lateral restraint shall be designed in such a way that the transverse deflection of the critical flange or chord at the point being restrained shall not exceed 0.005\(L\), where \(L\) is the span of the restrained member (Clause 5.8), when subjected to the force of 0.025\(P\) defined in Clause 3.3.4.2. This is equivalent to a minimum force extension stiffness of 100\(P\).

3.3.4.4 Multiple restraints. When more than one lateral restraint is used and the various restraints are uniformly distributed along the restrained member, then the force and stiffness requirements defined in Clauses 3.3.4.2 and 3.3.4.3 shall be distributed evenly amongst the various restraints; provided that the stiffness of an individual brace shall not be less than 4\(P\).
3.3.4.5 Parallel restrained members. When a series of parallel members are restrained by a line of restraints, the forces and stiffnesses determined by Clauses 3.3.4.2 and 3.3.4.3 shall be summed for all members and the restraints designed for the total force and stiffness so obtained. However, for seven or more parallel members, only the seven members with the largest P values need be considered.

3.3.4.6 Attachment of restraints. Lateral restraints shall be attached to a beam such that the force defined in Clause 3.3.4.2 can be transferred and such that the critical flange or chord deflection limit laid down in Clause 3.3.4.3 is not exceeded.

3.3.4.7 Critical flange or chord. The critical flange or chord of a member is that part which would deflect the furthest during buckling in the absence of the restraint being designed. If an exact analysis is not available this shall be determined as follows:

(a) When adjacent points of load application are restrained, the critical flange or chord of a beam between such points is the compression flange or chord.

(b) When a point of gravity load application is unrestrained, the critical flange or chord is the top flange.

(c) When the load is directly due to wind, the critical flange is the windward flange.

3.3.5 Deflection. Under the most adverse loading conditions, the deflections of neither a structure as a whole, nor any of its parts, shall be such as will impair the strength or serviceability of the structure or part, or lead to damage of other building components, or be unsightly. The responsibility for selecting deflection limits used in design shall rest with the Design Engineer.

Note: Guidance on maximum permissible deflections in specific cases is given in Appendix A.

3.3.6 Fatigue. For members subjected to frequent fluctuations of live load, the maximum permissible stress range shall be determined in accordance with Appendix B, as appropriate to the number of cycles, the class of constructional details adopted, and the materials used.

3.3.7 Corrosion Protection. Where steelwork in a structure is to be exposed to a corrosive environment, the steelwork shall be given adequate protection against corrosion, and the Design Engineer shall specify this protection. The degree of protection to be employed shall be determined after consideration has been given to use of the structure, climatic or other local conditions, and maintenance provisions.

Note: Recommendations for corrosion protection may be found in AS 2312 and SAA MAL3.

3.3.8 Brittle Fracture. Where steelwork in a structure is welded and parts subject to tensile stress may become liable to brittle fracture, an assessment of the required notch toughness of the fabricated steel may be made using the guidance given in Appendix D of AS 1554, Part 1.

NOTE: Welded structures of steel to AS 1204 are normally satisfactory but, under certain conditions, parts subjected to tensile stress may become liable to brittle fracture.

This type of failure should be considered possible where there is any combination of low temperatures and the following factors: thick plates or thick sections, induction in the ductility of the steel connections or details giving rise to severe stress concentrations; or the possibility of unattended defects in welding such as cracks or lack of fusion or root penetration.

Specific literature dealing with the problems of brittle fracture may also be consulted. (Refer to AWRA Technical Note 10 ‘Fracture Mechanics’.)
SECTION 4. GEOMETRICAL PROPERTIES

4.1 GENERAL. The geometrical properties of the gross and the effective cross-sections of a member or part of a member, shall be calculated on the following basis:

(a) The properties of the gross cross-section shall be calculated from the specified size of the member or part.

(b) The properties of the effective cross-section shall be calculated by deducting from the area of the gross section the following:

(i) The cross-sectional areas of excessive plate widths as given in Clause 4.3.

(ii) The sectional areas of all holes in the section, except that for parts in compression, a bolt or rivet hole shall not be a deduction. (See Clause 4.4.)

4.2 GEOMETRICAL PROPERTIES FOR CALCULATING BENDING STRESSES. It shall be permissible to calculate bending stresses in a beam or girder by using properties equivalent to those of the effective cross-section. Such equivalent properties shall be obtained by multiplying the properties of the gross cross-section by the ratio of the effective flange cross-sectional area to the gross flange cross-sectional area.

In welded construction the flange cross-sectional area shall be that of the flange plates alone.

In bolted or riveted construction, the cross-sectional area of the flanges shall be taken as the sum of the cross-sectional areas of the flange plates, flange angles, and the portion of the web and side plates (if any) between the flange angles.

4.3 PLATE THICKNESSES.

4.3.1 Plate and Flange Outstands. If the projection, b, of a plate or flange beyond its connection to a web, or other line of support or the like, exceeds the relevant values given in (a), (b), and (c) below, the area of the excess flange shall be neglected when calculating the effective geometrical properties of the section.

(a) Flanges and plates in compression with unstiffened edges... 2.5D[\(\sqrt{F_v}\)].

(b) Flanges and plates in compression with stiffened edges... 2D[\(\sqrt{F_v}\)] to the inner face of the stiffening.

Stiffened flanges shall include flanges composed of channel or I-sections or of plates with continuously stiffened edges.

(c) Flanges and plates in tension... 2D[\(\sqrt{F_v}\)].

In this Clause, \(D\) shall be taken to be the thickness of the plate, irrespective of whether the plate is a flange or other part of the member.

4.3.2 Flanges and Plates—Unsupported Widths. Where a plate is connected to other parts of a built-up member along lines generally parallel to the longitudinal axis of the member, the width, \(b_{\text{unsupported}}\), shall not exceed the following:

(a) For plates in uniform compression... 1400 \(\sqrt{F_v} [T_v]_1\).

However, where the width exceeds—

- 650 \(\sqrt{F_v} [T_v]_1\) for welded plates which are no stress-relieved, or
- 800 \(\sqrt{F_v} [T_v]_1\) for other plates,

the excess width shall be assumed to be located centrally, and its sectional area shall be neglected when calculating the effective geometrical properties of the section.

(b) For plates in uniform tension... 100 [\(T_v\)].

However, where the width exceeds 60 [\(T_v\)], the excess width shall be assumed to be located centrally, and its sectional area shall be neglected when calculating the geometrical properties of the section.

In this Clause, \(T_v\) shall be taken to be the thickness of the plate, irrespective of whether the plate is a flange or a web of the member.

4.3.3 Circular Hollow Sections. If a circular hollow section is used as a strut or a beam, the effective geometrical properties of the section shall be calculated using the specified outside diameter and a value for wall thickness which shall be the lesser of:

\[
\frac{3D^2}{640F_v[D/D]} + t
\]

where

\(D\) = the specified outside diameter of the section
\(t\) = the specified wall thickness of the section.


4.4 HOLES.

4.4.1 Effective Diameters. In calculating the area to be deducted for rivets, bolts or pins the diameter of the hole shall be used.

For countersunk holes the effective diameter of a hole shall be assumed to be the cross-sectional area of the hole, including the countersunk portion in the plane containing the axis of the hole, divided by the thickness of the material hole.

4.4.2 Combinations of Holes. Except as specified below, the area to be deducted from the gross cross-sectional area of a member, when making deductions for combinations of holes, shall be the sum of the sectional areas of the holes in that cross-sectional area right angles to the direction of stress in the member in which such sum is a maximum.

Where bolt or rivet holes are staggered, the area to be deducted shall be the sum of the sectional areas of all holes in a chain of holes extending progressively across the member, less \(\pi \sqrt{\pi/4}\) for each line extending between holes at other than right angles to the direction of stress, where \(\pi\), and \(\pi\) are respectively the staggered pitch, gauge and thickness associated with the line under consideration (see Fig. 4.4.2 (a)). The chain of lines shall be chosen to produce the maximum such deduction. For non-planar sections, such as angles with holes in both legs, the gauge \(g\), shall be the distance along the centre of the thickness of the section between hole centres (see Fig. 4.4.2 (b)).
4.5 Sectional Areas of Bolts, Screwed Tension Rods and Rivets.

4.5.1 Bolts and Screwed Tension Rods. The gross area of cross-section of a bolt or a screwed tension rod shall be taken as the area of cross-section of the body of the bolt or tension rod. The effective area of cross-section in tension shall be taken as the stress area.

The effective area of cross-section in shear shall be taken as the gross area of the cross-section or the core area as appropriate.

Note: Gross areas are calculated using a diameter equal to the shank diameter plus 0.5D/4 times the thread pitch and core area as tabulated in AS 1279, Table 1, column 17.

4.5.2 Rivets. The nominal diameter of a rivet shall mean the specified diameter cold before driving. The gross area of a rivet shall be taken to be the cross-sectional area of the rivet bolt as specified in Rule 4.4.1 excluding any counter sunk portion.

4.6 Maximum Slenderness Ratios. The maximum slenderness ratio of a beam, strut or tension member given in Table 4.6 shall not be exceeded. In this Clause, l is the effective length of the member (see Clause 5.9 and 6.3) and the radius of gyration, r, shall be based on the effective section defined in Clause 4.1.

<table>
<thead>
<tr>
<th>Member</th>
<th>Maximum Slenderness Ratio 1/l</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) A strut carrying compressive forces from dead load or from dead load and live load</td>
<td>180</td>
</tr>
<tr>
<td>(b) A discontinuous single-angle strut, single bolted or riveted</td>
<td>300</td>
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<tr>
<td>(c) A beam</td>
<td>300</td>
</tr>
<tr>
<td>(d) A strut carrying compressive forces resulting only from wind load</td>
<td>300</td>
</tr>
<tr>
<td>(e) A tension member carrying force from dead or live load where the minimum tensile force is less than 10 percent of the maximum tensile force</td>
<td>300</td>
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</tbody>
</table>

Note: The above limits may be exceeded for tension members if deflection stresses and vibrations are not excessive.
SECTION 5. DESIGN OF BEAMS

5.1 GENERAL. The calculated stress in a member shall not exceed any of the appropriate maximum permissible stresses given in Clauses 5.2, 5.3, 5.4 and 5.6 for bending, in Clause 5.10 for shear, and in Clause 5.11 for bearing, except that Clause 5.4 shall not apply in the following cases:

(a) a beam bent about its axis, of minimum strength;
(b) an I-beam or channel with equal flanges and with a value of $\frac{t_1}{t}$ less than 900/$\sqrt{F_{tv}}$;
(c) a rectangular hollow section beam with $R/D$ both greater than 0.15 (this includes hollow sections to AS 1163).

5.2 MAXIMUM PERMISSIBLE STRESS. The maximum calculated stress due to bending in a member shall not exceed the maximum permissible stress $F_{m}$ determined by formula 5.2(1) or 5.2(2):

For solid round and square bars, and for solid rectangular bars bent about the axis of minimum strength

$$F_{m} = 0.75F_{t}$$

For all other beams

$$F_{m} = 0.66F_{t}$$

$F_{t}$ as appropriate

5.3 MAXIMUM PERMISSIBLE COMPRESSIVE STRESS. The maximum calculated compressive stress due to bending in any part of a beam shall not exceed the maximum permissible stress $F_{m}$, which shall be the greater of

$$F_{m} = 0.60F_{m}$$

and the value determined by formula 5.3(1), 5.3(2), 5.3(3) or 5.3(4) as appropriate:

$$F_{m} = \left(0.72 - \frac{12}{256} \times \frac{b_{fl}}{t_{f}} \sqrt{F_{tv}} \right) F_{t}$$

5.4 MAXIMUM PERMISSIBLE STRESS IN A BEAM BENT ABOUT THE AXIS OF MAXIMUM STRENGTH.

5.4.1 Equal-flange I-beams or Channels. For an I-beam or channel with equal flanges bent about the axis of maximum strength, the maximum calculated stress shall not exceed the maximum permissible stress $F_{m}$ given in Tables 5.4.1(1), 5.4.1(2), 5.4.1(3), 5.4.1(4) for a yield stress $F_{y}$ of 250 or 350 MPa, or calculated from formula 5.4.1(1) or 5.4.1(2), as appropriate, and an elastic flexural-torsional buckling analysis.

In Tables 5.4.1(1), (2), (3) and (4) —

$T$ — the flange thickness as defined in Clause 1.6.

Note: Tables 5.4.1(1), (2), (3) and (4) have been derived from Clauses 5.4.1 and 5.5.

5.4.2 Laterally Unsupported Angle Sections. For a laterally unsupported angle section, the maximum stress $F_{m}$ may be calculated from formula 5.4.2(1) or 5.4.2(2) and an elastic flexural-torsional analysis.

Note: Alternatively, it is permissible to use either the approximate solutions or the tabulated load capacities for laterally unsupported angles in the "Safe Load Tables for Structural Steel" (published by The Australian Institute of Steel Construction).

5.4.3 Other Sections. The maximum calculated stress due to bending in a beam not otherwise covered by Clauses 5.4.1 and 5.4.2 shall not exceed the maximum permissible stress $F_{m}$ determined by formula 5.4.3(1) or 5.4.3(2), as appropriate:

$$F_{m} = \left[0.55 - \frac{10}{2F_{t}} F_{m} \right] F_{t}$$

Where $F_{m}$ is equal to or less than $F_{t}$

$$F_{m} = \left[0.95 - \frac{50}{F_{t}} \right] F_{m}$$

Where $F_{m}$ is equal to or greater than $F_{t}$

In formulas 5.4.3(1) and 5.4.3(2) above, the maximum stress $F_{m}$ in the beam at elastic buckling, shall be calculated in accordance with Clause 5.5 or by an elastic flexural-torsional buckling analysis.

$b_{u}$ = the unsupported width of the flange or plate between faces of support which is of thickness $T_{f}$ and in compression (see Clause 1.6 and Clause 4.3.2).

*b* See Appendix C.

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<th>Table 3.4.10.12</th>
<th>Table 3.4.10.13</th>
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**Table 3.4.10.12**

Maximum allowable stress in equal-flange beams or channels with $F' = 250$ MPa and $t > 20$ or $f > 85$.

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<th>$F' / mm$</th>
<th>$f / mm$</th>
<th>$t / mm$</th>
<th>$d / mm$</th>
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<td>190</td>
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**Table 3.4.10.13**

Maximum allowable stress in equal-flange beams or channels with $F' = 250$ MPa and $t > 20$ or $f > 85$.

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<th>$F' / mm$</th>
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Note: Values shown are valid only as determined by Clause [3].
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Note: Values above solid line may be reduced by Clause 5.3.

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Note: Values above solid line may be reduced by Clause 5.3.

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5.5 ELASTIC CRITICAL STRESS. If an elastic flexural-torsional buckling analysis is not carried out, the elastic critical stress $F_{cr}$ shall be determined by formula 5.5:

$$F_{cr} = K_f (A + K_d B)$$

For an I-beam with values of $T/I$ not greater than 2.0 and $d_1/d_2$ not greater than 1344 $\sqrt{v_F}$, the value of $F_{cr}$ shall be multiplied by a factor of 1.20.

In this clause (5.5),

$$A = B \left[ 1 + \left( \frac{D}{V} \right) \right]$$

$$B = \frac{26.5 \times 10^5 \text{MPa}}{G D^2}$$

For beams with a varying value of $G$, the value of $v_F$ at the point of maximum moment shall be used.

Notes:
1. Values of $A$ for various values of $d_1, d_2$, and $D/V$ and $D/I$ at various values of $B$ are given in Table 5.5.1.
2. For $T$ see Clause 5.3.1.
3. Guidance in calculating elastic buckling forces will be found in the relevant clause 9.1.4.

$c_t, c_l$ — respectively, the lesser and greater distances from the section neutral axis to the extreme fibres.

$K_f$ — a coefficient to allow for the reduction in thickness or breadth of the flanges between the points of effective lateral restraint. Values of $K_f$ are given in Table 5.5.2) for various values of $B$.

$N_c$ — the ratio of the total area of both flanges at the point of least bending moment to the corresponding area at the point of greatest bending moment between points of effective lateral restraint. The minimum value of $N_c$ shall be 0.25 and flange reductions shall be limited accordingly. The removal of a flange at the end of a beam shall be neglected when calculating $N_c$, provided that the length of flange removed does not exceed a length equal to the flange width at midspan.

$K_d$ — a coefficient to allow for the inequality of the flanges. Values of $K_d$ are given in Table 5.5.3) for various values of $M_p$.

$M_p$ — the ratio of the moment of inertia of the compression flange alone to the sum of the moments of inertia of the flanges, each calculated about its own axis parallel to the axis of minimum strength of the beam, at the point of maximum bending moment.

$T$ — the flange thickness as defined in Clause 1.6. Where the flanges are unequal, $T$ shall be calculated from the flange having the greater moment of inertia, calculated about its own axis parallel to the axis of minimum strength of the beam.

5.6 BENDING STRESSES FOR CASED BEAMS. If a beam is to be designed to take advantage of concrete encasement, the following conditions shall be satisfied:

(a) The section shall be of I-form with equal flanges and a single web, or of double open channel form with webs not less than 40 mm apart, and have a depth not exceeding 800 mm and a width not exceeding 550 mm overall.

(b) The beam shall be unpainted and be encased in concrete conforming to Grade 15, as defined in AS 1480.

(c) The minimum width of casing shall be equal to $(B + 100)$ mm, where $B$ is the overall width of the steel flange or flanges (mm).

(d) The outer faces and edges of the flanges of the beam shall have a concrete cover of not less than 50 mm.

(e) The casing shall be reinforced with wire complying with AS 1303. The diameter of the wire shall be at least 5 mm. Alternatively, the reinforcement shall be structural grade bars complying with AS 1362. The diameter of the bars shall be at least 6 mm. In either case, the reinforcement shall be in the form of stirrups or binding at not more than 150 mm pitch and so arranged as to pass through the centre of the covering to the flanges and supported by and attached to at least four longitudinal spacing bars.

The maximum calculated bending stress shall not exceed the lesser of:

(i) the value of any of the appropriate maximum permissible stresses permitted by Clauses 5.2, 5.3 and 5.4 in which the radius of gyration, $r_p$, shall be 0.2 $(B + 100)$ mm, and $T/B$ shall be as for the unceded section; and

(ii) 1.5 times that permitted for the uncased section.

Note: This Clause does not apply to box sections. See Clause 6.4 for girder beams.

5.7 PURLINS AND GIRTS. Sheeting shall be deemed not to restrain effectively the lateral deflection of the connected flange of a purlin or girt except in cases where experimental data are available to show that the sheeting and its fixings are able to provide such restraint and that twisting of the member will not occur.

Where special allowance for expansion is made in long-length sheeting, the fixings shall be deemed not to provide lateral support.

5.8 EFFECTIVE SPAN OF BEAMS. The effective span, $L$, of a beam shall be taken to be the distance between the centres of the supports, except for Clause 3.2.3 and where in Clause 6.4 the point of application of the reaction is taken as eccentric to the support, when it shall be permissible to take the effective span as the distance between the assumed points of application of the reactions.

For a member framing into a channel, the effective span shall be measured from:

(a) the shear centre of the supporting channel, or

(b) the distance between supports, whichever is greater.
### TABLE 5.5(1)

VALUES OF A AND B FOR CALCULATING $P_{om}$

where $A = B \left( \frac{1}{1 + \frac{1}{50} \frac{P_{om}}{f_{y}} \gamma_{m}} \right)$

$B = 26.5 \times 10^6$ MPa

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*See Note 1 to Clause 5.5.*

### TABLE 5.5(2)

VALUES OF $A_{1}$ FOR BEAMS WITH CURTAILED FLANGES

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### TABLE 5.5(3)

VALUES OF $A_{2}$ FOR BEAMS WITH UNEQUAL FLANGES

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5.9 EFFECTIVE LENGTH OF BEAMS FOR LATERAL BUCKLING.

5.9.1 General. In calculating the permissible stress for a beam, its effective length shall be determined by application of Clauses 5.9.3 to 5.9.6.

5.9.2 Restraints.

5.9.2.1 Torsional end-restraints. To provide restraint against torsion at the end of a beam, both flanges shall be fixed in position at the supports so as to prevent lateral movement.

Note: Torsional end-restraint may be achieved by—
(i) web or flange clamps;
(ii) load-bearing web stiffeners at supports (see Clause 5.9.2.4);
(iii) lateral-end frames or other external supports to the ends of the flanges; or
(iv) building into a wall.

Torsional end-restraint elements shall be designed to resist, in addition to wind and other applied external forces, a force at the support, positioned at the level of each flange, acting in a lateral direction, and having a value of at least 0.025 times the maximum force occurring in the flange.

5.9.2.2 Interim lateral restraints. To provide lateral restraint at an intermediate point in a beam, the critical flange (see Clause 3.3.4.7) shall be fixed in position so as to prevent lateral movement.

Members which provide intermediate lateral restraint to a beam shall comply with Clause 3.3.4.

5.9.3 Beams without Intermediate Lateral Restraints.

5.9.3.1 Restrained against torsion. Where each end of a beam is restrained against torsion, the effective length, \( l \), of the beam shall be taken to be:

(a) With ends of the critical flange unrestrained against lateral bending (i.e. free to rotate in plan at the supports)

\[ l = \text{span} \]

(b) With ends of the critical flange partially restrained against lateral bending (e.g. securely cleated flange connections)

\[ l = 0.85 \text{span} \]

(c) With ends of the critical flange fully restrained against lateral bending (i.e. not free to rotate in plan at the supports)

\[ l = 0.7 \text{span} \]

5.9.3.2 Partially restrained against torsion. Where each end of a beam is partially restrained against torsion (e.g. where the bottom flange is fixed in position and prevented from rotating about its longitudinal axis and the top flange is unrestrained), the values given in Clause 5.9.3.1 shall be multiplied by a factor of 1.20.

5.9.4 Cantilevered Beams without Intermediate Lateral Restraints. For a cantilevered beam of projecting length \( L \), the effective length, \( l \), of the cantilever shall be taken as follows:

(a) Fixed in position and direction at the support, free at the end

\[ l = 0.85 L \]

(b) Fixed in position and direction at the support, restrained against torsion at the end by contiguous construction (see Fig. 5.9.4 (a))

\[ l = 0.75 L \]

(c) Fixed in position and direction at the support, restrained against lateral deflection and torsion at the end (see Fig. 5.9.4 (b))

\[ l = 0.5 L \]

(d) Continuous at the support, unrestrained against torsion and free at the end (see Fig. 5.9.4 (c))

\[ l = 3.0 L \]

(e) Continuous at the support with partial restraint against torsion at the support and free at the end (see Fig. 5.9.4 (d))

\[ l = 2.0 L \]

(f) Continuous at the support, restrained against torsion at the support and free at the end (see Fig. 5.9.4 (e))

\[ l = L \]

Where in (d), (e) and (f) above, there is a degree of fixity at the free end, the effective length shall be multiplied by 0.750.85 and 0.50.85 for degrees of fixity corresponding to (b) and (c) above respectively.

5.9.5 Beams with Intermediate Lateral Restraint.

5.9.5.1 Lateral restraint at intervals. For a beam, including a cantilever, where lateral restraint of the critical flange is provided by members at intervals along the span, and where in addition the ends of the beam are restrained against torsion in accordance with Clause 5.9.2, the effective length, \( l \), of the beam shall be taken as the maximum distance, centre-to-centre, between the restraining members.

5.9.5.2 Continuous lateral restraint. For a beam, including a cantilever, where continuous lateral restraint of the critical flange is provided by a reinforced concrete deck or by a steel deck, and where in addition the ends of the beam are restrained against torsion in accordance with Clause 5.9.2, the effective length, \( l \), of the beam shall be taken as zero only when the deck construction is capable of resisting the lateral force, specified in Clause 3.3.4, in lateral flexure and shear.

5.9.6 Beams with Critical Flange Loading Unrestrained Laterally. For a beam, including a cantilever, where the load is applied to the critical flange, and the load and the flange are free to move laterally but the ends of the beam are restrained against torsion, the values for the effective length, \( l \), given in Clause 5.9.3 and the relevant parts of Clause 5.9.4 shall be multiplied by a factor of 1.20.

5.10 SHEAR.

5.10.1 Maximum Shear Stress. The calculated maximum shear stress in a member shall not exceed the value of \( F_{\text{m}} \), determined by formula 5.10.1:

\[ F_{\text{m}} = 0.45 F_{\text{r}} \]

5.10.2 Average Shear Stress in Rolled I-beams and Channels, Plate Girders, Box-sections, Rectangular and Circular Hollow Sections. The average shear stress, calculated on the effective sectional area (see Clause 5.10.6) of a rolled I-beam, a rolled channel, a plate girder, a box-section, or a rectangular or circular hollow section shall not exceed either value of \( F_{\text{m}} \) given in (a) and (b) below, as appropriate:

(a) For an unstiffened web, the lesser of the values determined by formulae 5.10.2(a) and 5.10.2(b):

\[ F_{\text{m}} = 0.45 F_{\text{r}} \]

\[ F_{\text{m}} = 0.45 F_{\text{r}} \]

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Fig. 5.9.4 CANTILEVER BEAMS

(a) Cantilever built-in at support, restrained against torsion at the end

(b) Cantilever built-in at support, restrained laterally at end

(c) Cantilever \( l_0 \) continuous at support, unrestrained against torsion at the support and unrestrained at the end

(d) Cantilever \( l_0 \) continuous at support, partially restrained against torsion at the support and unrestrained at the end

(e) Cantilever span continuous at support, fully restrained against torsion at the support and unrestrained at free end
\[ F_a = 0.37 F_e \cdot \frac{\sqrt{F_e}}{F_e}  \]
\[ F_s = 0.37 F_e \cdot \frac{1.3}{\left( 1 + 0.5 \sqrt{\frac{a}{b}} \right)} \]
\[ \frac{F_a}{F_s} = \frac{1}{1 + 0.5 \sqrt{\frac{a}{b}}} \]

where
\[ a = \text{the greater clear dimension of the web panel} \]
\[ b = \text{the lesser clear dimension of the web panel} \]

5.10.3 Shear Stresses in Other Sections. A section not covered by Clause 5.10.2 shall comply with Clause 5.10.1. Such a section shall include at least one whose web varies in thickness or is penetrated by large openings.

5.10.4 Effective Sectional Area. The effective cross-sectional area, \( A_e \), resisting shear in a member shall be as defined in (a), (b), and (c) below, provided that the shear-resistant part specified is not penetrated by openings larger than 700 mm² or rivet holes:

(a) **Rolled I-beams and channels** . . . the product of the thickness of the web and the full depth of the section.
(b) **Plate girders and box girders** . . . the product of the thickness of the web or webs and the full depth of the web plate.
(c) **Circular hollow sections** . . . 0.60 times the gross cross-sectional area.

5.11 BEARING STRESSES.

5.11.1 Maximum Permissible Stress. The bearing stress in any part of a beam, except at a hole in a bolted connection, when calculated on the net projected area of contact, shall not exceed the value of \( F_b \) determined by formula 5.11:

\[ F_b = 0.75 F_e \]

5.11.2 Dispersion of Force through Flange to Web. For any section, other than hollow sections to AS 1163, \( F_{bw} \) where a force is directly applied to a flange, it shall be considered as dispersed uniformly through the flange at the flange-web intersection at an angle of 30 degrees to the surface of the flange.

5.12 FLANGE DETAILS.

5.12.1 Flange Splices.

5.12.1.1 Butt welds. Where a butt weld is used to effect a splice in a flange, the plates shall be prepared in accordance with AS 1554 and joined by a continuous complete-penetration butt weld which will develop the full strength of the smaller plate.

5.12.1.2 Cover plates. Where cover plates are used to effect a splice in a flange, the sum of their areas shall be at least 1.05 times the area of the smaller flange plate spliced. If the centroid of the cover plates does not coincide with the centroid of the flange, provision shall be made in the design of the plates for this eccentricity. The connections on each side of the splice shall be capable of transmitting at least the greater of—

- 1.05 times the computed force in the flange, and
- 0.50 times the maximum safe force in the smaller flange.

5.12.2 Curtailment of Flange Plates. If a flange plate is curtained, it shall be extended beyond the point at which calculations show that it is no longer required. The extended portion shall be fastened to the beam by sufficient fillet welds, high-strength bolts or rivets to develop the calculated force in the plate at the forepoint of the beam. The beam section used in the calculations shall include the curtained plate.

If fatigue is a design criterion, any change in a flange section shall be brought about by a gradual transition (see Appendix D).

5.12.3 Connection of Flanges to Web. A flange of a plate girder shall be connected to the web by sufficient welds, bolts or rivets to transmit the horizontal shear force combined with any vertical forces arising from loads which are applied directly to the flange.

However, in welded construction, vertical forces causing direct compression shall be deemed to be resisted by the bearing between the flange and the web only when the web is machined or otherwise prepared to the Supervising Engineer’s satisfaction, and is in close contact with the flange before welding.

Where the web is not so prepared, vertical forces shall be transmitted by welds and the welds shall be designed accordingly.

5.13 WEB DETAILS.

5.13.1 Web Plates.

5.13.1.1 Minimum thickness. The thickness of a web plate shall not be less than the following:

(a) For an unstiffened web . . . \( d/180 \).
(b) For a vertically stiffened web . . . the greater of—

- \( 1/180 \) of the smaller clear panel dimension, and \( d \sqrt{F_e} \)
- \( 1/120 \) of the greater clear panel dimension, and \( d \sqrt{F_e} \)

(c) For a web stiffened both vertically and horizontally, with a horizontal stiffener at a distance from the compression flange equal to 0.4 times the distance from the compression flange to the neutral axis . . . the greater of—

- \( 1/180 \) of the smaller clear panel dimension, and \( d \sqrt{F_e} \)
- \( 1/120 \) of the greater clear panel dimension, and \( d \sqrt{F_e} \)

When there is also a horizontal stiffener at the neutral axis . . . the greater of—

- \( 1/180 \) of the smaller clear panel dimension, and \( d \sqrt{F_e} \)
- \( 1/120 \) of the greater clear panel dimension, and \( d \sqrt{F_e} \)
### TABLE 5.10.2(1)
**MAXIMUM PERMISSIBLE AVERAGE SHEAR STRESS, \( F_s \) (MPa), IN A STIFFENED WEB FOR A STEEL WITH \( F_y = 250 \) MPa**

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### TABLE 5.10.2(2)
**MAXIMUM PERMISSIBLE AVERAGE SHEAR STRESS, \( F_s \) (MPa), IN A STIFFENED WEB FOR A STEEL WITH \( F_y = 340 \) MPa**

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5.13.2 Web panel — maximum dimension. In a stiffened girder, the greater clear dimension of a web panel shall not exceed 2700.

5.13.3 Splices in webs. A splice in the web of a plate girder or rolled section used as a beam shall be designed to resist the shearing forces and moments in the web at the spliced section.

5.13.4 Side reinforcing plates. Where two additional plates are provided to augment the strength of a web, they shall be placed on each side of the web and shall be equal in thickness to each other. Alternatively, if a single plate is provided for this purpose on one side of the web only, proper account shall be taken of the lack of symmetry.

The proportion of shear force assumed to be resisted by such a plate or plates shall be limited by the amount of horizontal shear which it can transmit to the flanges through its fastenings, and the plate and its fastenings shall be carried beyond the point at which calculations show that it becomes unnecessary. The extended portions of the plate shall be attached to the beam by fasteners sufficient to develop the proportion of the shear force which it resists at the foregoing point.

5.13.2 Load-bearing Web Stiffeners.

5.13.2.1 All sections. For any section, load-bearing stiffeners shall be provided at a point of concentrated force or reaction where such force or reaction exceeds the value of $F_{u}/B$ —

$$F_{u} = \text{the maximum permissible axial stress for struts as defined in Clause 6.1 for a slenderness ratio equal to } (d/\sqrt{3})/h$$

$$B = \text{the length of the stiff portion of the bearing, plus the additional length given by disposition at } 45 \text{ degrees to the level of the neutral axis, plus the thickness of flange plates at the load point. The stiff portion of the bearing is that portion of the length which cannot deform appreciably in bending, and shall not be taken as greater than half the depth of the beam for points of support at the ends of a simply supported beam, and the full depth of a beam in any other case.}$$

5.13.2.2 Plate girders. In addition to the requirements of Clause 5.13.2.1, load-bearing stiffeners shall be provided also at the supports where either —

(a) the web is overstressed in shear (see Clause 5.13.1.6(b)); or

(b) the web would be otherwise overstressed in bearing (see Clause 5.13.1) or in the web connections (see Clause 9.4).

5.13.2.3 Design for concentrated force. Load-bearing stiffeners shall be designed as struts assuming the section to consist of the pair of stiffeners together with a length of web loaded symmetrically about the plane of the stiffeners and equal, where available, to 40 times the web thickness. The radius of gyration shall be taken about the axis parallel to the web of the beam, and the calculated stress shall not exceed the appropriate maximum permissible value for a strut determined by assuming an effective length equal to 0.7 times the length of the stiffener.

The force assumed in design shall be the concentrated force or reaction and any eccentricity of this force with respect to the centroid of the assumed strut section shall be considered in design.

The outstanding legs of each pair of load-bearing stiffeners shall be so proportioned that the bearing stress on that part of their area in contact with the flange and clear of the root of the flange or flange angles, or clear of the flange welds, does not exceed the bearing stress specified in Clause 5.11. Load-bearing stiffeners shall be provided with sufficient welds, high-strength bolts, or rivets to transmit to the web the whole of the concentrated force.

A load-bearing stiffener shall be fitted to provide a tight and uniform bearing upon the loaded flange, unless welds are provided between the flange and stiffeners for the purpose of transmitting the concentrated force or reaction. Where a point of concentrated force is directly over a support, this provision shall apply to both flanges.

5.13.2.4 Design for torsional end-restraint. If load-bearing stiffeners are the sole means of providing a torsional end-restraint (Clause 5.9.2.1), the second moment of area, $I_w$, of the pair of load-bearing stiffeners about the centreline of the beam web shall not be less than

$$\frac{PY}{125 W}$$

where $T = \text{the maximum thickness of the critical flange}$

$R = \text{the reaction of the beam at the support}$

$W = \text{the total load on a beam between supports}$

And in addition, the stiffeners and their bases in conjunction with the bearing of the girder shall be capable of resisting a moment due to the forces specified in Clause 5.5.2, acting at the centroid of the critical flange.

5.13.3 Intermediate Web Stiffeners for Plate Girders.

5.13.3.1 Vertical stiffeners. Where vertical stiffeners are used to satisfy the requirements of Clause 5.10 and/or Clause 5.13.1.1, their spacing shall not exceed 1.5 times the web thickness.

Vertical stiffeners shall be designed so that the second moment of area, $I_w$, of a pair of stiffeners, or of a single stiffener used independently, is not less than

$$1.5 \left( \frac{d^4 + d^2}{S^2} \right)$$

where

$S = \text{the longitudinal spacing between the centres of successive vertical stiffeners in a beam}$

Note. $I_w = \text{the second moment of area (moment of inertia) of the cross section of a pair of web stiffeners about the centroid of a beam web, or of a single stiffener about the face of a beam web, as appropriate.}$

5.13.3.2 Horizontal stiffeners. If the thickness of a vertically stiffened web plate is less than $(d/\sqrt{3})/3200$ (see Clause 5.13.1.1(b)), horizontal stiffeners shall be provided as follows:

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(a) A horizontal stiffener, on one or both sides of
the web, shall be placed at a distance from the
compression flange equal to 0.4 times the
distance from the compression flange to the
neutral axis, and the stiffener (single or double)
shall have a moment of inertia, $I_w$, not less
than 4500.

(b) A second horizontal stiffener, on one or both
sides of the web, shall be placed on the neutral
axis of the girder where the thickness of the
web plate is less than $d_W P_c/9000$ (see Clause
5.13.1.1(c)), and the stiffener (single or double)
shall have a moment of inertia, $I_w$, not less
than $d_0 p$.

5.13.3.3 External forces on intermediate stiffeners. If an intermediate vertical stiffener is sub-
jected to a bending moment or shear force due to
eccentricity of vertical loads, or the action of trans-
verse forces, the moment of inertia of the stiffener
given in Clause 5.13.3.1 above shall be increased as
follows:

(a) Bending moment on stiffener due to eccen-
tricity of vertical loading with respect to the
vertical axis of the web —

$$I_e = 1.5 \frac{MP}{H}$$

(b) Lateral loading on stiffeners —

$$I_e = \frac{3}{2} \frac{Q t^2}{E t}$$

where

$M =$ the applied bending moment

$Q =$ the transverse force to be taken by the
stiffener and deemed to be applied at the
compression flange of the girder.

5.13.3.4 Connection of intermediate stiffeners to
web. If an intermediate vertical or horizontal stiffener
not subjected to external loads is used to stiffen a
web, the stiffener shall be connected to the web by
sufficient welds, high-strength bolts or rivets to with-
stand a shearing force, between each component of
the stiffener and the web, of not less than —

$$125 \frac{t}{b} \text{ kN/m}$$

where

$t =$ the web thickness, in millimetres

$b =$ the outstand of the stiffener, in millimetres.

For a stiffener subjected to external forces the shear
between the web and stiffeners due to these forces shall
be added to the above value.

5.13.3.5 Outstand of all web stiffeners. The outstand of a load-bearing or other stiffeners from a
web shall not exceed the requirements of Clause
4.3.1 (a) and (b), as appropriate.

5.14 SEPARATORS AND DIAPHRAGMS. If
separators or diaphragms are to be used to permit
two or more rolled T-section beams or channels
placed side by side to act together as a unit in the
distribution of external loads between them, the
separators and diaphragms shall meet the following
requirements:

(a) Separators, made up of spacers and through
bolts, shall not be used to transmit forces
between the beams, other than those due to
transverse forces (if any) and a transverse
force, $Q$, taken as not less than 0.025 times the
maximum force occurring in the most heavily
loaded compression flange of any member
forming the unit. The transverse force, $Q$, shall
be taken as shared equally between the
separators.

(b) Diaphragms shall be used where external
vertical as well as transverse forces are to be
transmitted from one beam to another. The
diaphragms and their fastenings shall be pro-
portioned to distribute the forces applied to
them and in addition to resist the transverse
force, $Q$, specified above, and resulting shear
forces. The transverse force, $Q$, shall be taken
as shared equally between the diaphragms.

(c) Nevertheless, in the case of grillage beams
encased in concrete, the maintenance of correct
beam spacing only shall be necessary.
SECTION 6. DESIGN OF STRUTS

6.1 AXIAL STRESSES IN UNCASED STRUTS.

6.1.1 Struts Loaded Concentrically. Where a strut is concentrically loaded by an axial force, the average compressive stress, \( f_{x0} \) (MPa), calculated on the effective cross-sectional area of the strut, shall not exceed the value given in Table 6.1.1 for the appropriate yield stress. For a steel of yield stress not provided for in Table 6.1.1, the value of \( f_{x0} \) shall be calculated from formula 6.1.1:

\[
F_{x0} = \frac{1}{D} \left( \frac{F_x + \eta + \sqrt{\frac{1}{2} F_x^2 - F_{x0}^2}}{2} \right) - F_{x0},
\]

6.1.1

\( l/r \) = slenderness ratio = the ratio of the effective length to appropriate radius of gyration (see Clause 6.3 and Section 4).
\( \Omega \) = load factor, taken as 1.06 for the purpose of this Clause.
\( \eta = 0.0003 \frac{f_{y}}{f_{x0}} \)
\( f_{x0} \) = Euler critical stress = \( \frac{f_{y}^2}{E} \)

6.1.2 Built-up Struts. A strut composed of two or more components shall be designed as a single member in accordance with Clause 6.1.1, only when it satisfies the relevant requirements of Clauses 6.6 and 6.7. Otherwise, each component shall be designed as a separate member.

6.1.3 Slender-leg Struts. When the value of \( R/T \) for any angle exceeds 200/\( \sqrt{f_{y}} \), the calculated stress shall not exceed the lesser of:

(a) 0.50 \( f_{x0} \), and
(b) the value of \( f_{x0} \) calculated from Clause 6.1.1.

6.2 AXIAL FORCES IN CASED STRUTS. If a strut is to be designed to take advantage of concrete encasement, the following conditions shall be satisfied:

(a) The section shall be of I-form or of double open-channel form with the webs in contact or not less than 40 mm nor more than half their depth apart. The cross-sectional dimensions of the steel strut shall not exceed 300 x 300 mm, with the web or webs parallel to the 800 mm dimension. The slenderness ratio of the encased section, determined by taking the effective length, \( l \) as the distance centre-to-centre of lateral restraints, shall not exceed 250.
(b) When channels or axles they shall be laced or battened, or connected together if in contact, to comply with Clause 6.6 or 6.7, or with the relevant rules of AS 1554, as appropriate.

(c) The strut shall be unainted and be encased in concrete conforming to one of the standard grades of structural concrete in AS 1480.
(d) The minimum width of casing shall be equal to \( 2B + 100 \) mm, where \( B \) is the overall width of the steel flange or flanges (mm).
(e) The outer faces of the flanges of the steel sections shall have a concrete cover of not less than 50 mm.
(f) The casing shall be reinforced with wire complying with AS 1302. The diameter of the wire shall be at least 5 mm. Alternatively, the reinforcement shall be structural grade bars complying with AS 1302. The diameter of the bars shall be at least 6 mm. In either case, the reinforcement shall be in the form of stirrups or binding at not more than 150 mm pitch and so arranged as to pass through the centre of the covering to the flanges and supported by and attached to at least four longitudinal spacing bars.

When calculating the maximum permissible axial force on a cased strut, an equivalent steel area shall be used equal to

\[
A_s + 0.52 A_s = A_s
\]

where

\( A_s = \) the gross cross-sectional area of a concrete-encased strut neglecting any casing in excess of 75 mm from the overall dimensions of the steel section and neglecting any applied finish
\( A_s = \) the effective cross-sectional area of a steel member

The maximum permissible axial stress, \( f_{x0} \) on the equivalent strut section shall be determined by Clause 6.1, in which \( f \) shall be determined by Clause 6.3 and \( f \) about the centroidal axis in the plane parallel to the web or webs shall be taken as 0.5 \( (B + 100) \) mm. The radius of gyration about the other axis shall be taken as that of the encased section.

Nevertheless, the maximum permissible axial force on a cased strut shall not exceed 2.0 times nor be less than 1.0 times that which would be permitted on the uncased section.

6.3 EFFECTIVE LENGTH OF STRUTS.

6.3.1 General. The slenderness ratio of a strut shall be calculated as the ratio of the effective length, \( l \) to the appropriate radius of gyration, \( r \). The effective length, \( l \), shall be derived from the actual strut length, \( L \). The actual strut length shall be taken as the length from centre-to-centre of intersections with supporting members, or the cantilevered length in the case of free-standing struts.

Where a rational elastic buckling analysis is required by this Clause, such an analysis will be deemed to have been carried out when the effective length of a strut is chosen by an appropriate method given in Appendix E.

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6.3.2 Sideways Prevented. In a frame where in-plane stiffness is provided and in a truss, the effective length, \( L \), of a strut shall be assumed to be equal to the unbraced strut length, \( L_u \), unless a smaller value is calculated by a rational elastic buckling analysis.

Notes: Elastic stiffness in a frame may be provided by diagonal bracing, or by shear walls, or by floor slabs or roof decks secured horizontally to walls or bracing systems parallel to the plane of buckling of the frame.

Alternatively, for a strut of a steel whose yield stress does not exceed 280 MPa, the effective length shall be as given below for the cases indicated:

(a) In a rectangular framework where both ends of the strut are effectively fixed in position and restrained in direction —

\[ I = 0.75L \]

(b) In a rectangular framework where the strut is effectively fixed in position at both ends and restrained in direction at one end only —

\[ I = 0.85L \]

(c) In a truss —

\[ I = \text{the reduced value for the effective length of the strut given in Paragraph E2 of Appendix E, as appropriate.} \]

6.3.3 Sideways Not Prevented. The effective length, \( L \), of a compression member in a frame which depends on lateral action to limit sway, shall be calculated by a rational elastic buckling analysis and in no case shall be less than the actual unbraced length, \( L_u \) (See Appendix E).

6.4 ECCENTRICITY FOR STRUTS.

6.4.1 Location of Beam Reaction. For the purpose of Clause 3.2.2, a beam reaction or a similar load on a strut shall be taken as acting at a distance of 100 mm from the face of the strut towards the span or at the centre of bearing, whichever gives the greater eccentricity, except as qualified below.

(a) For a strut cap, the load shall be taken as acting at the appropriate face of the strut or edge of packing, if used, towards the span of the beam.

(b) For a roof truss bearing on a cap, it shall be permissible to neglect eccentricity in simple details and connections not capable of developing a significant moment.

6.4.2 Continuous Struts. For a continuous strut complying with Clause 6.5, the bending moment due to eccentricity of loading at any one floor or horizontal frame level shall be taken as follows:

(a) Ineffective at the floor or frame levels above and below that floor.

(b) Divided between the strut lengths above and below that floor or frame level in proportion to the relevant values of \( I/\beta \) of the strut, where \( I = \text{second moment of area of cross-section.} \)

6.5 SPLICES.

6.5.1 Ends of Struts Prepared for Full Contact. If the ends of two butting lengths of a strut are required to be in full contact (see Clause 11.2.4), the lengths shall be connected so that they are secured in line for all transverse forces and moments and so that any resultant tension is resisted.

6.5.2 Ends of Struts Not Prepared for Full Contact. If the ends of two butting lengths of a strut are not required to be in full contact, the connections joining the lengths shall be designed to transmit at least all the force to which they are subjected.

6.5.3 Arrangement of Splikes. If two butting lengths of a strut are joined so that the centroids of the connections do not coincide with the centroids of the lengths joined, provision shall be made in the design for the resulting eccentricity.

6.5.4 Minimum Forces. A splice or end-connection of a strut shall be designed for any applied moment and the greater of:

(a) 0.50 times the maximum permissible compressive force in the member; and

(b) 1.05 times the compressive force in the member.

6.6 STRUTS WITH TWO OR MORE MAIN COMPONENTS IN CONTACT. If a strut is formed from plates, rolled sections or hollow sections, directly connected in any combination, the connections shall comply with Section 9. The longitudinal spacing and proportions of fasteners provided shall be adequate for the transfer of calculated stress in all cases.

6.7 STRUTS WITH TWO SEPARATE COMPONENTS.

6.7.1 Design Forces for Connections. If a strut composed of two or more main components is intended to act as a single member, the connections between the components shall be proportioned to resist the stresses arising from a lateral force, \( Q \), applied at any point on the length of the member in the most unfavourable direction, together with 1.10 times the stresses arising from external forces and bending moments.

The force, \( Q \), shall be not less than 0.025 times the total axial force in the strut. The resulting forces for lacing, and forces and moments for battens and other connections, shall be considered as divided equally among connection planes parallel to the direction of force (see Clause 6.7.2 (b) (vi) and Clauses 6.7.3, 6.7.4 and 6.7.5).

6.7.2 Struts Composed of Two Components Back-to-back. If a strut composed of two angles, channels or tees, back-to-back, it is intended to act as a single member, the two components shall comply with subclause (a) or (b) below, as appropriate.
(a) Components shall be in contact and shall be connected so as to comply with Clause 6.6.

(b) Components shall be separated by a distance not greater than 50 mm and shall comply with the following:

(i) The components shall be connected at the ends and at intermediate points so that the maximum slenderness ratio, \( \ell / r \), of each component shall exceed neither 50 nor 6.6 times the maximum slenderness ratio of the strut as a whole, where \( \ell \) is the maximum centre-to-centre distance between consecutive connections. This distance shall not exceed one-third the length of the member between end connections.

(ii) The ends of the components shall be connected together with two bolts or rivets, or their equivalent in welding.

(iii) Where connections are made by welding, solid packings shall be such as will allow effective fillet welds to be used.

(iv) Where connections are made by bolting or riveting, the bolt or rivet shall pass through solid washers or packing. Where the legs of the connected angles or tabs of connected tees are 125 mm or over, or where the webs of connected channels are 150 mm wide or over, not less than two bolts or rivets, located transversely, shall be used in each connection. It shall be permissible to use staggered bolts or rivets through tabs of connected tees less than 125 mm wide.

(v) Bolts or rivets shall be at least

- for components up to and including 10 mm thick . . . 16 mm diameter
- for components over 10 mm thick up to and including 16 mm thick . . . 20 mm diameter
- for components over 16 mm thick . . . 24 mm diameter.

(vi) The strut shall not be subjected to transverse forces in a plane perpendicular to the welded, bolted or riveted surface.

6.7.3 Laced Struts. A laced strut, intended to act as a composite member, shall comply with the following requirements:

(a) General.

(i) Except for tie plates as specified in sub-clause (c) below, a lacing system shall not be combined with members or diaphragms transverse to the longitudinal axis of the member unless all forces resulting from any consequent deformation of the component members are calculated and are provided for in the design of the lacing and its fastenings.

(ii) Lacing shall be provided in the form of flat bars, or rolled sections, or hollow sections of at least equivalent strength and shall comply with subclause (b) below.

Note: The lacing system should preferably not vary throughout the length of the strut and should be connected to effect the minimum practicable interruption of the transverse or the system.

(b) Details.

(i) The slenderness ratio, \( \ell / r \), of a lacing element shall not exceed 140.

The effective length, \( l \), of a lacing element in welded, non-alip bolted or riveted construction shall be taken as the length between the inner end-connecting welds or fasteners of the element to the main components for single lacing, or 0.7 times this length for double lacing effectively connected at the intersections.

(ii) Where a flat lacing bar is mortised bolted or riveted, its minimum width shall be as follows:

- for 24 mm diameter fasteners . . . 67 mm
- for 20 mm diameter fasteners . . . 59 mm
- for 16 mm diameter fasteners . . . 51 mm

(iii) The angle of inclination of lacing to the longitudinal direction of the member shall lie within the following limits:

- single lacing . . . 50 degrees to 70 degrees inclusive
- double lacing . . . 40 degrees to 50 degrees inclusive.

(iv) The slenderness ratio of a main component between points of attachment of lacing members shall be not greater than 50 or 0.6 times the maximum slenderness ratio of the strut as a whole, whichever is the lesser, where the effective length shall be taken to be the length between centres of the consecutive connections of the lacing elements to the components and the radius of gyration shall be taken to be the minimum radius of gyration of the main component of the member.

(v) Where welded elements overlap the main components, the amount of lap shall be not less than

- four times the thickness of the overlapping bar or leg, as appropriate, or
- four times the mean thickness of the flange of the main member to which they are attached, whichever is the lesser.

The welding shall be provided at least along each side of the connection for the full length of the lap, and in members subjected to reversal of stress the welding shall be returned along the ends of the connection for a length equal to at least four times the thickness of the overlapping bar or leg.

If lacing is fitted between the main components the lacing shall be connected to each member by continuous profile fillet welds or full-penetration butt welds and shall be positioned in line with the flanges of the main components.

(c) Tie plates.

(i) A laced strut shall be provided with tie plates at the ends of the lacing system, at points where the lacing system is interrupted, and where the strut is connected to another member. End tie plates shall have a width measured along the axis of
the member of not less than the perpendicular distance between centroids of their connections to the main components. Intermediate tie plates shall have a width of not less than three-quarters of this distance.

(ii) A tie plate and its connections shall comply with Clause 6.7.4 (b) (iii) and (iv) and shall be treated as a batten for design purposes.

(iii) The thickness of a tie plate shall be not less than 0.02 times the distance between the innermost lines of welds, bolts or rivets, except when the tie plate is effectively stiffened at the free edges, in which case the edge stiffeners shall have a slenderness ratio not greater than 170.

6.7.4 Battened Struts. A battened strut, intended to act as a single member, shall comply with the following requirements:

(a) General. Battens shall be placed opposite each other at each end of the strut and at points where the strut is restrained laterally. In addition, the number of battens shall be such that the strut is divided into not less than three approximately equal parts. Battens shall consist of plates, channels, or I-sections, and shall be welded, non-slip bolted, or riveted transversely to the main components.

Next: Battened struts should have two main components of the same cross section so arranged that their axes of maximum strength are co-planar. The battens should be spaced and proportioned uniformly throughout.

(b) Design details.

(i) The slenderness ratio of a main component between battens shall exceed neither 50 nor 0.6 times the maximum slenderness ratio of the member as a whole. For calculating the slenderness ratio of the main component the effective length shall be taken to be the distance centre-to-centre of the consecutive end fastenings of adjacent battens and the radius of gyration shall be taken to be the minimum radius of gyration of the smaller main component, if any.

(ii) A battened plate and its connections shall be designed to transmit simultaneously a longitudinal shear force equal to \( \frac{S}{2} \) and a moment equal to \( \frac{Q}{2} \) to the main components —

where

- \( S \) = the longitudinal distance centre-to-centre of battens
- \( d \) = the minimum transverse distance between the centroids of the end connections of the batten to the main components
- \( Q \) = the transverse shear force specified in Clause 6.7.1
- \( N \) = the number of parallel planes of battens.

The maximum spacing of connections shall not exceed the relevant requirements of Section 9.

(iii) The effective length of a batten plate along the axis of the member shall be taken to be the longitudinal distance between end fastenings. The effective length of an end batten shall be not less than 0.8 times the perpendicular distance between the centroids of the main components. The effective length of an intermediate batten shall not be less than 0.7 times the perpendicular distance between the centroids of the main components.

The effective width of any batten shall be not less than —

where the main components are of equal size . . . 2.0 times the width of a main component in the plane of the battens

where the main components are not of equal size . . . 2.0 times the width of the smaller main component in the plane of the battens.

(iv) Battens shall have a thickness of not less than 0.02 times the minimum distance between the innermost lines of connecting welds or bolt or rivet groups, except when the batten plates are effectively stiffened at the free edges, in which case the edge stiffeners shall have a slenderness ratio about the axis parallel to the strut axis not greater than 170.

(v) The length of weld connecting each longitudinal edge of a batten element to each main component shall in the aggregate be not less than half the effective length of the batten element and at least one-third of the weld shall be placed at each end of the longitudinal edge. In addition, the welding shall be returned along the transverse edges of the element for a length equal to at least four times the thickness of the element or web, as appropriate (see Clause 3.3.6). Where a batten is fitted between the main members, it shall be connected to each component either by fillet welds on each side of the plate at least equal in length to that as previously specified or by complete-penetration butt welds along the whole width of the element. A batten shall be positioned in line with the flanges of the main components. Fillet welds shall in no case be less than 5 mm.

(c) Other battened struts. A strut not complying with clauses (a) and (b) above, or a strut subjected in the plate of the battens to eccentricity of loading, applied moments, or lateral forces, shall be designed according to an adequate theory of elastic stability using a load factor of not less than 1.70, or 1.3 if appropriate (see Clause 3.3.5). Alternatively, when proved by test, a strut shall satisfy Clause 3.5.

6.7.5 Starred Angles. A battened strut composed of two angles forming a cruciform cross-section and intended to act as a single member shall comply with Clause 6.7.4 (b) except as follows:
The intermediate battens shall be in pairs placed in contact against each other unless they are welded to form cruciform battens. A transverse shear force of $O_2/2$ shall be taken as occurring separately about each rectangular axis of the whole member.

A longitudinal shear force of $O_2/2$ and a moment $O_2$ shall be taken with respect to each batten in each of two planes, except where the maximum value of $I/r$ can occur about a rectangular axis. In the latter case each batten shall be designed to resist a shear force of 0.025 times the total force.

$Q, S,$ and $a$ are as defined in Clauses 6.7.1 and 6.7.4 (b) (3), as appropriate.

6.8 CAPS AND BASES FOR STRUTS.

6.8.1 Concentric Forces. If a rectangular plate is loaded concentrically, its minimum thickness, $T$, shall be determined by formula 6.8.1:

$$T = \sqrt{\frac{3F_a}{f_e \left( a - \frac{b}{4} \right)}} \quad 6.8.1$$

where $a$ = the greater projection of the plate beyond the strut

$b$ = the lesser projection of the plate beyond the strut

$f_e$ = the calculated pressure on the underside of the baseplate

$F_a$ = maximum permissible bending stress for baseplates

$= 0.75 F_v$

If gussets are used for transmitting forces to the baseplate, the projecting distances, $a$ and $b$, shall be measured from the extremities of the gussets only when the gussets are designed for the resulting forces.

6.8.2 Eccentric Forces and Non-rectangular Plates. If a baseplate will not distribute a force uniformly or if the baseplate is not rectangular, calculations shall be carried out to determine the bending stresses. The maximum permissible bending stress in a baseplate shall be 0.75 $F_v$.

6.8.3 Connection to Bases. If a baseplate is not prepared for full contact to comply with Clause 11.2.4, the connections shall be designed to transmit all the axial force to the baseplate.

6.8.4 Encased Girder Beams. If girder beams are fully encased in concrete and requirements (a), (b) and (c) below are satisfied, the maximum permissible stresses for the beams shall be 1.33 times those permitted for unencased beams given in this standard.

(a) The concrete shall conform to Grade 15, as defined in AS 1480.

(b) The beams shall be spaced to provide a gap of at least 75 mm between the edges of adjacent flanges and shall be effectively held in position.

(c) The thickness of concrete cover provided at any point on the girder beams shall be at least 100 mm.

Notwithstanding Clause 3.3.1, the calculated stress in a girder beam shall not exceed the maximum permissible stress determined by this Clause.

6.9 BEARING STRESSES. The bearing stress in any part of a strut, except at a hole associated with a bolted connection, when calculated on the net projected area of contact, shall not exceed the value of $F_b$ determined by formula 5.11.

The bearing stress associated with a bolt, rivet or pin shall comply with Clause 9.5.
SECTION 7. DESIGN OF TENSION MEMBERS

7.1 AXIAL STRESSES IN TENSION MEMBERS.
In an axially loaded tension member, the average tensile stress calculated on the effective sectional areas (see Clause 4.1) shall not exceed —

\[ F_{tu} = 0.60 F_{u} \]

7.2 TENSION MEMBERS SUBJECTED TO BENDING. A tension member subjected to bending shall comply with Clause 8.2.2 except where Clause 7.5.3 (a) is applied.

7.3 DISTRIBUTION OF FORCES.

7.3.1 End Connections Providing Uniform Force Distribution. Where, for design purposes it is assumed that tensile forces are distributed uniformly to a tension member, the end connections shall satisfy the following requirements:

(a) Connections shall be made to each part of the member and shall be symmetrically placed about the centroidal axis of the member.

(b) Each part of the connection shall be proportioned to transmit at least the maximum force carried by the connected part of the member.

7.3.2 End Connections Providing Non-uniform Force Distribution. If the end connections of a tension member do not provide uniform distribution of force in the member, the following requirement shall be satisfied, as appropriate:

(a) Eccentrically-connected angles, channels and tees. If eccentricity of force is neglected in an eccentrically-connected angle, channel or tee, the area used for calculating the average tensile stress in the member shall be determined by (i) to (v) below:

(i) For a single angle connected through one leg, the effective cross-sectional area of the angle shall be taken as —

\[ A_e = \left( 1 + \frac{A_t}{A_t} \right) \]

where

\[ A_t = \text{the effective cross-sectional area of the connected leg (see Clause 4.1)} \]

\[ A_t = \text{the gross cross-sectional area of the unconnected leg} \]

The gross cross-sectional area of a leg of an angle shall be taken to be —

\[ \frac{t (a - \frac{t}{2})}{\frac{a}{2}} \]

where

\[ B = \text{the length of the outer face of the leg} \]

\[ t = \text{the thickness of the leg} \]

(ii) For two angles, back-to-back and connected to one side of a gusset plate or section that is positioned across the back of the angles, the effective cross-sectional area of each angle shall be taken as —

\[ A_e = \left( 1 + \frac{A_t}{A_t + A_t} \right) \]

where \( A_t \) and \( A_t \) are as defined in (i) above.

(iii) For a single channel connected through the web, the effective cross-sectional area of the channel shall be taken to be as determined from the expression in (ii) above, where

\[ A_e = \text{the effective cross-sectional area of the web (see Clause 4.1)} \]

\[ A_t = \text{the gross cross-sectional area of both flanges} \]

The gross cross-sectional area of the web shall be taken to be \( Bt \) and that of a flange shall be taken to be \( B(t - t) \), where

\[ B, D, t \] and \( t \) are as defined in Clause 1.6.

(iv) For a single tee connected through its table, the effective cross-sectional areas of the tee shall be taken to be as determined from the expression in (ii) above, where

\[ A_e = \text{the effective cross-sectional area of the table (see Clause 4.1)} \]

\[ A_t = \text{the gross cross-sectional area of the stem} \]

The gross cross-sectional area of the table shall be taken to be \( BT \) and that of the stem shall be taken to be \( (D - t)T \), where

\[ B = \text{the width of the table} \]

\[ D = \text{the depth of the section measured parallel to the stem} \]

\[ T = \text{the average thickness of the table} \]

\[ t = \text{the thickness of the stem} \]

(v) For a member composed of two angles, channels or tees placed back-to-back and connected to each side of a gusset plate or part of a rolled section, the axial force shall be deemed not to be applied eccentrically only if the components are connected together as required by Clause 7.4.3. In such a case the area of cross-section of the member shall be the sum of the areas of the effective cross-sections of each component.

(b) Symmetrical I-sections or channels connected by both flanges. For a symmetrical rolled or built-up member or section 1 or channel form connected by both flanges only, provision shall be made for the effect of non-uniform distribution of force to the cross-section.

In the case of fillet-welded, bolted or riveted connections the effective cross-sectional area, as determined by Clause 4.1, shall be multiplied by a factor equal to \( \left( 1 - cL \right) \).

The value of \( c \) shall be determined by considering the shape formed by that portion of the section between the neutral axis and the
outer face of the flange; \( e \) is the distance from the centroid of this shape to the connected face.

\( L \) is the length between the first and last rows of fasteners in the connection or, when the member is welded, \( L \) is the length of longitudinal weld provided to each side of the connected legs.

Each flange connection shall be proportioned to transmit at least half the maximum load in the member.

7.4 TENSION MEMBERS WITH TWO OR MORE MAIN COMPONENTS.

7.4.1 General. A tension member composed of two or more components intended to act as a single member shall comply with Clauses 7.4.2 to 7.4.5 inclusive.

7.4.2 Design Forces for Connections. If a tension member is composed of two or more main components, the connections between the components shall be proportioned to resist the stresses arising from the external forces and bending moments (if any). The forces for lacing bars, and forces and moments (if any) for battens, shall be considered as divided equally among connection planes parallel to the direction of force.

7.4.3 Tension Members Composed of Two Components Back-to-back. A tension member composed of two flats, angles, channels or tees, connected back-to-back either in contact or separated by a distance not exceeding 50 mm, shall comply with the following requirements:

(a) Where the components are separated, they shall be connected together at regular intervals in their length by welding, bolting or riveting, so that the slenderness ratio between connections does not exceed 300.

(b) Where the components are separated, they shall be connected through solid washers or packings to comply with Clause 6.7.2 (b) (iii) and (iv).

(c) Where component members are in contact back-to-back, they shall be connected together as required by Clause 9.6.3 or the relevant requirements of AS 1554, as appropriate.

7.4.4 Lacing of Tension Members. A tension member composed of two components connected by lacing shall comply with the following requirements:

(a) The lacing of the tension member shall comply with Clause 6.7.3 (a).

(b) The requirements of Clause 6.7.3 (b) shall be satisfied except that:

(i) the slenderness ratio of the lacing shall not exceed 210;

(ii) the slenderness ratio of a main component between lacing points shall not exceed 300.

(c) For tie plates and equivalent bracing, the requirements of Clause 6.7.3 (c) shall be satisfied except that:

(i) the thickness of tie plates shall be not less than 0.017 times the distance between the innermost lines of connections for unstiffened plates;

(ii) the requirements of Clause 9.6.3 shall govern tie plate connections.

7.4.5 Battening of Tension Members. A tension member composed of two components connected by battens shall comply with the following requirements:

(a) The battening of a tension member shall comply with Clause 6.7.4 (a).

(b) The battening of a tension member shall comply with Clause 6.7.4 (b) except that —

(i) the spacing of battens shall be such that the maximum slenderness ratio of each main component does not exceed 300;

(ii) battens attached by bolts or rivets shall be symmetrically placed at least three bolts or rivets and Clause 6.7.4 (b) (ii) and (iv) shall not apply;

(iii) batten plates shall have a thickness of not less than 0.017 times the distance between the innermost lines of connections for unstiffened plates;

(iv) intermediate battens shall have a width of not less than half the effective width of end batten plates.

(c) A battened tension member not complying with the foregoing requirements, and those subjected, in the plane of the battens, to eccentricity of force, applied moments, or lateral forces, shall comply with Clause 6.7.4(c).

7.5 CONNECTIONS.

7.5.1 Minimum Connections. Welded, bolted and riveted connections at the ends of tension members shall be designed for at least the maximum force determined from the following:

(a) 0.50 times the maximum permissible force in the member;

(b) 1.00 times the tensile force in the member.

7.5.2 Splices. A butt-welded splice in a tension member shall be designed to develop the full strength of the member. A splice formed in any other way shall be designed for at least the maximum force determined from the following:

(a) 0.50 times the maximum permissible force in the member;

(b) 1.05 times the tensile force in the member;

(c) 25 kN.

Where cover plates are not disposed to suit the distribution of stress in the member or where both surfaces of the parts spliced are not covered, proper provision shall be made for the resultant eccentricity. The fasteners connecting a cover plate to a member at a splice shall be designed to develop at least the maximum load in the cover plate.

7.5.3 Pin Connections. A pin connection in a tension member shall comply with the following requirements:

(a) The average stress across the minimum net sectional area through the pin transverse to the axis shall not exceed 0.45\(f_{pu}\);

(b) The longitudinal net sectional area beyond the pin hole on the reaction side parallel to the axis of the member shall not be less than the minimum permissible net area, \(A_\text{m} \), of the tension member.
(c) In the case of a member without stiffened edges, either full length or adjacent to the pin hole, the value of the individual plate thickness, \( t_p \), measured adjacent to the pin hole shall be greater than \( \sqrt{A/12} \).

(d) The thickness of pin plates shall be such as will make up the total effective area to at least 1.33:4, adjacent to the pin hole. The length of the first pin plate on each side of the member, measured from the centre of the pin to the end on the reaction side, shall be equal to at least the width of the plate, and such width shall be as wide as the dimensions of the member will allow. Successive plates may be reduced in width and length only when all the pin plates are connected with sufficient fastenings to transmit at least the stress induced in them by the pin.

If pin plates cannot be arranged to avoid eccentricity, proper account of such eccentricity shall be taken in their design.

7.6 BEARING STRESSES. The bearing stress in any part of a tension member, when calculated on the net projected area of contact, shall not exceed the value of \( F_b \) determined by formula 5.11.

The bearing stress associated with a bolt, rivet or pin shall comply with Clause 8.5.
SECTION 8. COMBINED STRESSES

8.1 GENERAL. A member subjected to a combination of direct stresses shall be proportioned to comply with Clause 8.3 for any load or any load combination specified in Clause 3.3.1.

8.2 INDIVIDUAL MOMENTS AND FORCES. The member shall be proportioned so that the calculated stresses in it due to moments and forces, considered individually, shall not exceed the relevant maximum permissible stresses specified in this standard.

8.3 DIRECT STRESS COMBINATIONS.

8.3.1 Axial Compression and Bending. Members subjected to both axial compression and bending shall be proportioned to satisfy the following requirements:

(a) For the member as a whole and using the maximum values of \( f_{max} \) and \( f_{min} \):

\[
\frac{f_{max}}{f_{max}} + \frac{C_{max}}{C_{max}} \leq 0.6 \left( 1 - \frac{f_{min}}{f_{max}} \right) F_{min}
\]

where

\[
C_{max} = \frac{f_{max}}{f_{min}}
\]

However, if \( f_{min}/f_{max} \) is less than 0.15, the following simplified expression may be used:

\[
\frac{f_{max}}{f_{max}} + \frac{C_{max}}{C_{max}} \leq 0.1 \quad \ldots \quad 8.3.1(a)(b)
\]

The values of \( f_{max} \) and \( f_{min} \) to be used in the above formulae shall be the lesser of the values of the maximum permissible stresses, \( f_{max} \) and \( f_{min} \), given in Section 5 for bending about the appropriate axis.

(b) At a support and using the values of \( f_{max} \) and \( f_{min} \) at the support:

\[
\frac{f_{max}}{f_{max}} + \frac{C_{max}}{C_{max}} \leq 0.6 \left( 1 - \frac{f_{min}}{f_{max}} \right) F_{min}
\]

where

\[
C_{max} = \frac{f_{max}}{f_{min}}
\]

For the purpose of this Clause:

- \( F_{min} \) = lesser maximum permissible axial compressive stress about either principal axis obtained using Section 6

- \( f_{max}, f_{min} \) = calculated compressive bending stress defined as follows:

  For expressions 8.3.1 (a) (1) and (2), \( f_{max} \) and \( f_{min} \) are maximum values of bending stress about an axis of bending occurring either at or between braced points.

  For expressions 8.3.1 (b), \( f_{max} \) and \( f_{min} \) are values of bending stress at the support.

\( f_{max}, f_{min} \) = maximum permissible compressive bending stress about each axis of bending, obtained using Section 5

For expressions 8.3.1 (a) (1) and (2), \( f_{max} \) and \( f_{min} \) are derived using Clauses 5.2, 5.3 and 5.4 (if applicable).

8.3.2 Axial Tension and Bending. A member subjected to both axial tension and bending shall be proportioned so as to satisfy the following requirements:

\[
\frac{f_{max} + f_{min}}{0.6F_{t}} \leq 0.6 \left( 1 - \frac{f_{min}}{f_{max}} \right) F_{min}
\]

where

\[
C_{max} = \frac{f_{max}}{f_{min}}
\]

Notes:

1. \( C \) is the ratio of the smaller to the larger bending moment at any point of the member.

2. \( E \) is the modulus of elasticity of the member.

3. For members in frames where bending is un resisted by other members or other supports, the value of \( C \) may be determined by analytical means. In the absence of such means, the following values may be used:

   - For members where ends are restrained
     \[
     C_{E} = 0.85
     \]
   - For members whose ends are un restrained
     \[
     C_{E} = 1.0
     \]

   For expressions 8.3.1 (b), \( F_{t} \) is to be used in the calculation.

   Where

   - \( r_{E} \) = the effective length in the plane of bending determined in accordance with Clause 6.3
   - \( C_{E} \) = the corresponding radius of gyration.

4. For the purposes of this Clause, for a column, where an allowance is made for the force carried by the concrete in accordance with Clause 5.3, the ratio \( f_{max}/f_{min} \) is to be determined at the ratio or the calculated axial force on the column to the maximum permissible axial force determined by Clause 6.3.
For the purpose of this Clause —  
\[ \frac{f_{uix}}{f_{u}} + \frac{f_{uex}}{f_{u}} \leq 1.0 \]  
8.3.3 Biaxial Bending. A member subject to biaxial bending in the absence of any axial force shall be proportioned so as to satisfy at all points —  

where  
\[ f_{uix}, f_{uex} \]  
calculated compressive bending stress about each axis of bending at the point under consideration.
SECTION 9. DESIGN OF CONNECTIONS

9.1 MINIMUM DESIGN FORCE ON CONNECTIONS. Connections carrying calculated stresses, except connections of lacing, tag rods, pull-ins and girts, shall be designed for a minimum force or reaction of 25 kN.

9.2 CHOICE OF FASTENERS. Where slip under load must be avoided in a connection, high-strength bolts or friction-type joint (see AS 1531), welding or rivets shall be used.

9.3 COMBINED CONNECTIONS. When non-slip fasteners (such as high-strength bolts in a friction-type joint, welds or rivets) are used in a connection in conjunction with slip-type fasteners (such as mild steel bolts or high-strength bolts in bearing-type connections), all the load shall be assumed to be carried by the non-slip fasteners.

However, when welding is used in a connection in conjunction with other non-slip fasteners, the following restrictions shall also apply:

(a) When welds are used in conjunction with rivets, all the load applied after welding shall be assumed to be carried by the welds.

(b) Any load initially applied directly to the welds shall not be assumed to be distributed to fasteners added after the application of the load.

9.4 CONNECTION STIFFENERS. The design of web stiffeners to carry the flange forces from some transverse in framing member, at a connection shall be in accordance with either Clause 5.1.1 and 5.1.3.2 or Clause 10.10.2. If Clause 10.10.2 is used, the force to be transferred by the weld connecting the stiffener to the web may be taken as 0.6°FwAw.

9.5 STRESSES IN BOLTS, SCREWED TENSION RODS, RIVETS AND PINS.

9.5.1 Forces on Bolts and Rivets. The force on an individual bolt or rivet in a group of bolts or rivets due to an eccentric force or simultaneous moment shall be calculated using an elastic method of analysis.

9.5.2 Maximum Permissible Stresses. For any bolt complying with Clause 5.1.1 and not tightened in accordance with the methods specified in AS 1531, for any rivet, the calculated tensile, Fσ, shear, Fs, and bearing, Fb, stresses calculated on the effective area specified in Table 9.5.2 shall not exceed the respective maximum permissible stresses Fσ, Fs and Fb, defined in Table 9.5.2.

For bolts complying with AS 1252 and tightened in accordance with the methods specified in AS 1531, the maximum permissible stress shall be in accordance with AS 1531.

9.5.3 Combined Stresses. When a bolt or rivet is subjected to a combination of tensile, Fσ and shear, Fs, stresses, the expression —

\[
\left(\frac{F_{\sigma}}{F_{\sigma}}\right)^{1} + \left(\frac{F_{s}}{F_{s}}\right)\]

shall not exceed unity. This Clause shall not apply to bolts to AS 1252, tightened in accordance with AS 1531.

9.6 DESIGN DETAILS FOR FASTENERS.

9.6.1 Minimum Pitch. The distance between centres of fastener holes shall be not less than 2.5 times the nominal diameter of the fastener. However, if it is intended to tighten bolts with a special tensioning tool, the minimum distance between the centres of bolt holes shall be appropriate to the type of tool used.

Note: The minimum pitch may also be affected by Clause 9.5.2.

<table>
<thead>
<tr>
<th>Type of stress</th>
<th>Maximum permissible stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension, Fσ, on the effective area (see Clause 4.5.1)</td>
<td>Fσ is the smaller of 0.6(Fσ) or 0.45(Fσ)</td>
</tr>
<tr>
<td>Shear, Fs, on the effective area (see Clause 4.5.1)</td>
<td>Fσ is the smaller of 0.35(Fσ) or 0.25(Fσ)</td>
</tr>
<tr>
<td>Bearing, Fb, on an area of plate given by the product of the plate thickness and the bolt shank diameter</td>
<td>Fb = 2.1(Fσ)</td>
</tr>
</tbody>
</table>

Note:
1. Fσ and Fb are the yield and tensile strengths of the fastener and Fc is the yield stress of the plate subjected to bearing.
2. The use of these stress values when using bolts in steel hatted connections containing more than five bolts in a line is not recommended.
3. The reduction in these stress values to account for non-uniform distribution of force is not recommended. Buildings methods for reducing these stresses are recommended. Buildings, Part B—Design Details of Steel Connections, published by the Australian Institute of Steel Construction.
9.6.2 Minimum Edge Distances.

9.6.2.1 General. The minimum distances from the centre of a fastener to the edge of a plate or the flange of a rolled section shall be as specified in Table 9.6.2.

<table>
<thead>
<tr>
<th>Sheared or load flame cut edge</th>
<th>Rolled plate / Machine flame cut, saw or slotted edge</th>
<th>Rolled edge of a rolled section</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.75 (d_i)</td>
<td>1.50 (d_i)</td>
<td>1.25 (d_i)</td>
</tr>
</tbody>
</table>

Notes: \(d_i\) = nominal diameter of fastener, in millimetres.

9.6.2.2 Minimum edge distance in direction of component of force. The minimum distance \(s\) from the centre of a fastener to the edge of a plate or section measured in the direction of a component of force on the fastener shall not be less than that provided in Clause 9.6.2.1 nor less than —

\[
P = \frac{F_y}{t}\]

where —

\(P\) = component of force acting on the edge of the hole in the direction of the minimum distance toward an edge, in kilonewtons
\(F_y\) = yield stress of the plate or section, in megapascals
\(t\) = thickness of the plate or section, in millimetres

The edge of a plate or section shall be deemed to include the edge of an adjacent fastener hole for the purposes of this Clause.

9.6.3 Maximum Pitch. The maximum distance between centres of fasteners shall be the lesser of 14\(d_i\) and 200 mm. However, in the following cases the maximum distances shall be:

(a) For fasteners which are not required to carry calculated force in regions not liable to corrosion, the lesser of 32\(d_i\) and 300 mm.
(b) For an outside line of fasteners in the direction of major force, the lesser of 4\(d_i\) + 100 mm, and 200 mm.

Where two lines of load-carrying rivets are staggered, an equal interval, and the transverse pitch does not exceed 75 mm, the maximum distance between centres of rivets along each line shall be 1.5 times the appropriate values given above.

9.6.4 Maximum Edge Distance. The maximum distance from the centre of any fastener to the nearest edge of parts in contact with one another shall be 17 times the thickness of the thinnest outer connected part under consideration, but shall not exceed 150 mm.

9.6.5 Locking of Nuts. Where a mild steel bolt is subjected to severe vibration or to tensile force, the nut shall be effectively locked in position after tightening.

9.6.6 Long-grip Rivets. The grip of a rivet carrying calculated force shall not exceed eight times the diameter of the rivet hole, and where the grip exceeds six diameters, the number of rivets required by calculations shall be increased by not less than 0.6 percent for each additional millimetre of grip.

9.7 DESIGN DETAILS FOR PINS.

9.7.1 General. Where a pin is used to allow free rotation at a support, adequate provision shall be made for the effective lubrication of the pin in service.

9.7.2 Bending Stresses in Pins. In calculating the bending stress in a pin, forces shall be taken as applied along the mid-thickness line of the load-carrying elements bearing on it.

9.8 WELDS.

9.8.1 General. Welds shall comply with AS 1554 and the following Clauses 9.8.2 to 9.8.4. Attention shall also be given to possible failure by fatigue or stress concentration.

9.8.2 Maximum Permissible Stresses in Welds. If two components are joined by welding, the maximum permissible stress in the weld shall be as given in subclauses (a) and (b) below.

(a) Butt welds. The relevant value of \(F_y\) to be used in calculating maximum permissible stresses for complete-penetration butt welds shall be the minimum value pertaining to the components joined. For incomplete-penetration butt welds, \(F_y\) shall be taken as 0.50\(F_{nom}\), where \(F_{nom}\) is the nominal tensile strength of the electrode used.

(b) Fillet welds. The maximum permissible shear stress in a fillet weld shall be 0.33\(F_{nom}\), where \(F_{nom}\) is the nominal tensile strength of the electrode used. The stress in the weld shall be calculated on the basis of the maximum design throat thickness specified in AS 1554. Where a weld is subjected to a combination of stresses, the equivalent stress given by —

\[\sqrt{f(t^2 + 3t)}\]

shall not exceed \(0.33F_{nom}\) or 0.57\(F_{nom}\), where \(f\) is the resultant direct stress normal to the throat thickness and \(t\) is the vector sum of the shear stresses in the plane of the throat thickness. Any longitudinal direct stress in the fillet weld may be ignored.

Notes: Recommended electrodes for steel are given in AS 1556, Part 1, Table 4.3.1 (a).

9.8.3 Butt Welds.

9.8.3.1 Continuous incomplete-penetration butt welds. An incomplete-penetration butt weld shall be permitted only in a strait in which the weld is not to be subjected to impact forces or force reversals other than those due to wind. The weld shall be continuous.

9.8.3.2 Intermediate complete-penetration butt welds. An intermediate complete-penetration butt weld shall be used only in a part of a structure not subjected to fatigue loading and shall be positioned to minimize eccentricity of force. For calculation purposes a length, \(l\), shall be subtracted from weld lengths for each stop and start position. The minimum effective length of any such weld shall not be less than 4\(l\), with a minimum of 40 mm, and the longitudinal space between effective lengths of such welds shall be not more than 12\(l\).

In this Clause, \(l\) is the design (effective) throat thickness of a complete-penetration butt weld as defined in AS 1554.
9.8.4 Fillet Welds.

9.8.4.1 Transverse spacing. If two parallel fillet welds connect two components along the joint lines in the direction of stress to form a built-up member, the transverse distance between the welds shall not exceed $\frac{3a}{2}$, except that in the case of intermittent fillet welds at the ends of a tension member, the transverse distance shall not exceed either $\frac{16a}{3}$ or 200 mm.

It shall be permissible to use fillet welds in slots and holes in the direction of stress to satisfy this Clause.

9.8.4.2 Intermittent fillet welds—general. Except at the ends of a built-up member, the clear spacing between the lengths of consecutive colinear intermittent fillet welds shall not exceed the lesser of—

(a) for elements in compression . . . $16a$, and 300 mm
(b) for elements in tension . . . $24a$, and 300 mm

The minimum effective length of a segment of intermittent fillet weld shall be the greater of—

(i) 40 mm; and
(ii) four times the nominal size of the weld.

An intermittent fillet weld shall not be used where the space between consecutive welds will permit corrosion to occur in inaccessible areas.

9.8.4.3 Intermittent fillet welds, built-up members. If intermittent fillet welds connect components forming a built-up member, the welds shall comply with the following requirements:

(a) At the ends of a tension or compression component of a beam, or at the ends of a fully stressed tension member, when side fillets are used alone they shall have a length along each joint line at least equal to the width of the connected component. If the connected component is tapered, the length of weld shall be the greater of—

(b) At the cap plate or baseplate of a strut, welds shall have a length along each joint line of at least the maximum width of the strut at the contact face.

(c) Where a beam is connected to the face of a strut, the welds connecting the strut components shall extend between the levels of the top and bottom of the beam and in addition—

for an unrestrained connection . . . a distance $D$ below the lower face of the beam;
for a restrained connection . . . a distance $D$ above and below the upper and lower faces of the beam.

In this subclause, $D$ is the maximum cross-sectional dimension of the strut.

9.9 PACKING.

9.9.1 Bolts or Rivets Through Packing. Where bolts or rivets through a packing plate are required to carry calculated shear, the number of fasteners required, determined by calculations, shall be increased by 1.2 percent for each millimetre thickness of packing in excess of 6 mm. For a double shear connection with packing on both sides of a member, the increase shall be determined by the thickness of the thicker packing provided.

9.9.2 Packing in Welded Construction. Where packing is welded between two members and is less than 6 mm thick, or is too thin to allow provision of adequate welds or to prevent buckling, the packing shall be trimmed flush with the edges of the stress-carrying element and the size of the welds along the edges shall be increased over the required sizes by an amount equal to the thickness of the packing.
If the "plastic design method" is used for the design of a structure or part of a structure which is within the scope of this standard, the method shall only be used if—

(i) the section used is not a cold-formed hollow section;
(ii) the yield stress of the steel does not exceed 360 MPa;
(iii) the stress-strain characteristics of the steel are not significantly different to that typically obtained for steel to AS 1204.

The load factor shall not be less than 1.0.60 for any load or any combination of loads specified in Clause 3.3.1.

The requirements of this standard with respect to maximum permissible stresses shall be waived for this method. However, the design shall comply with all other relevant provisions of this standard.

Notes:
1. Most of the requirements of Section 10 apply only to double symmetric I sections. The use of other sections may require supporting experimental or theoretical evidence.
2. References to the plastic design method are to be found in: BCSCA® Publication No. 29—Plastic Design. AISCE® Manual No. 41—Commentary on Plastic Design in Steel.

10.2 BEAMS. The calculated maximum moment capacity, \( M_{pu} \), of a beam shall be

\[
M_{pu} = S F_a
\]

where \( S \) is the plastic modulus of the section.

10.3 TENSION MEMBERS. The calculated maximum load capacity, \( P_{pu} \), of a tension member shall be

\[
P_{pu} = A F_a
\]

10.4 STRUTS. The calculated maximum load capacity, \( P_{pu} \), of a strut shall be

\[
P_{pu} = A F_a \times 0.60
\]

where \( P_{pu} \) is the maximum permissible stress given in Clause 6.1 using an effective length \( l \) equal to the actual length \( l \) (see Clause 6.3).

10.5 MEMBERS SUBJECTED TO COMBINED BENDING AND AXIAL FORCE.

10.5.1 General. The calculated maximum moment capacity of a member subjected to combined bending and axial force shall be reduced below the value given in Clause 10.2 where \( P/P_{pu} \) exceeds 0.15. The reductions shall be as specified in Clauses 10.5.2 and 10.5.3 below.

10.5.2 Moment Capacities. The calculated maximum moment capacity of a member where \( P/P_{pu} \) exceeds 0.15 shall be \( M_{pu} \) where \( M_{pu} \) is the plastic moment reduced by axial force. For rolled I-sections specified in AS 1123 add about 90% of the axial of maximum strength \( M_{pu} \), may be calculated from

\[
M_{pu} = 1.18 \left( 1 - \frac{P}{P_{pu}} \right) M_a
\]

10.5.3 Struts.

10.5.3.1 Slender struts. A member for which the ratio \( P/P_{pu} \) exceeds both 0.15 and the value of \( P/P_{pu} \) given by

\[
\frac{P}{P_{pu}} = 1 + \frac{\beta}{\gamma}
\]

shall not be assumed to contain plastic hinges although it shall be permissible to design the member as an elastic part of a plastically designed structure. Such a member shall be designed according to the maximum permissible stress requirements given elsewhere in this standard. However, when designing such a member for factored loads, the maximum permissible stresses shall be increased by the same factor.

In this clause

\[
\lambda = \frac{l}{r} \sqrt{\frac{E}{F_a}}
\]

\( r \) is the radius of gyration about the same axis as the applied moment
\( \beta \) is the ratio of end-moments, each measured in the same rotational direction and chosen with the numerically larger moment in the numerator (i.e. \( \beta \) ranges from 1 to 4 for double curvature, through 0 for one end pinned, to -1 for single curvature)
\( I \) is the actual strut length (see Clause 6.3).

10.5.3.2 Stubby struts. A member subjected to combined bending and axial compression with \( P/P_{pu} \) exceeding 0.15 and not coming within the scope of Clause 10.5.3.1 shall have a reduced moment capacity of \( M_{pu} \), where \( M_{pu} \) is as given in Clause 10.5.2.

10.5.4 Low Load-ratio Members. A member assumed to contain plastic hinges and subjected to combined bending and axial compression with \( P/P_{pu} \) not exceeding 0.15 shall have a value of \( \lambda \) not exceeding the value given by

\[
\lambda \leq \frac{P}{P_{pu}} = 0.60 + 0.40 \beta
\]

where \( \lambda \) and \( \beta \) are as defined in Clause 10.5.3.1.

10.6 SHEAR. The calculated maximum shear capacity, \( V_{pu} \), of a beam or a beam-column shall be

\[
V_{pu} = 0.55 A F_a
\]

10.7 STABILITY. The elastic buckling load of a plastically designed frame or a frame containing plastically designed components, shall be at least 3.00 times the plastic collapse load. An accurate estimate of the elastic buckling load is not available, this Clause shall be deemed to be satisfied for frames of up to three stories if the compressive force, \( P \), in each member does not exceed

\[
P = 0.33 r F_a
\]

for buckling in any direction, where \( r \) is determined by Clause 6.3.

For frames of over three stories, the calculated plastic collapse load shall include an assessment of the moment caused by the possible combination of high axial force and transverse deflection. (See Appendix A, Paragraph A1.3.1(a).)

* British Constructional Steelwork Association.
† American Society of Civil Engineers.
10.8 MINIMUM THICKNESSES.

10.8.1 Compression Outstands. A flange or other compression element required to participate in a plastic hinge action shall not project beyond its outermost point of attachment by more than 136t_f/\sqrt{F_v}.

For the purposes of this Clause, web stiffeners at plastic hinges shall be proportioned as compression elements.

10.8.2 Unsupported Widths. The distance between adjacent parallel lines of attachment of a compression flange or another compression element to other parts of member, where such flanges or elements are required to participate in a plastic hinge action, shall not exceed 512t_f/\sqrt{F_v}.

10.8.3 Webs in Shear. If the depth d_i of a web subject to shear and required to participate in a plastic hinge action exceeds 688t_f/\sqrt{F_v}, then the compressive axial force, P_v, on the member shall not exceed

\[ P_v \left( \frac{0.70 - \frac{d_i}{t} - \sqrt{t}}{1600} \right) \]

The maximum permissible value of d_i in any plastic hinged zone shall be 1120t_f/\sqrt{F_v}.

10.9 LATERAL RESTRAINTS. For the purpose of this Clause, M_d shall be taken as M_{ed} or M_{m}, as appropriate.

If the length along the member in which the applied moment exceeds 0.85M_{ed} is less than 640t_f/\sqrt{F_v}, at least one critical flange support shall be provided within or at the end of this length and the spacing of the adjacent supports shall not exceed 960t_f/\sqrt{F_v}.

If the length along the member in which the applied moment exceeds 0.85M_{m} is greater than or equal to 640t_f/\sqrt{F_v}, then the critical flange shall be supported in such a manner that no portion of this length is unsupported for a distance of more than 640t_f/\sqrt{F_v}.

Lateral restraints for the remaining elastic portions of the member shall be designed in accordance with Section 5 or 6, as appropriate, using stresses derived from the plastic bending moments divided by 1/0.60.

In this Clause a may be taken as unity or calculated by the following expression:

\[ a = \frac{1.5}{\sqrt{(1 + R^2)}} \]

where R is the ratio of the rotation at the hinge point to the relative elastic rotation of the far ends of the beam segment containing the plastic hinge.

Note: The lateral restraints provided by this Clause will ensure that a section delivers its full moment and deformation capacity. This may be too great for some design circumstances.

With the approval of the Building Authority, the Design Engineer may use the methods described in Note 2 to Clause 10.1 which allow a reduced amount of resistance to be used of partially plastic sections subject to tension and bending. It is essential to use widely accepted means and that any associated reductions in moment and deformation capacity are fully considered in the design.

10.10 WEB STIFFENING.

10.10.1 Excessive Shear Forces. Web stiffeners or doubler plates shall be provided when the requirements of Clause 10.6 are not met, in which case the stiffeners or doubler plates shall be capable of carrying that portion of the force which exceeds the shear capacity of the web.

10.10.2 Concentrated Loads. Web stiffeners shall be provided at points on a member where the concentrated force delivered by the flanges of another member framing into it will produce web crippling opposite the compression flange or high tensile stress in the connection of the tension flange. This requirement shall be deemed to be satisfied if web stiffeners are placed:

(a) opposite the compression flange of the other member when

\[ t < \frac{A_s}{F_v + \frac{k}{4} t} \]

(b) opposite the tension flange of the other member when

\[ T_r < 0.4 t / A_t \]

where

- t = thickness of web to be stiffened
- k = distance from outer face of flange to web toe of fillet of member to be stiffened
- P_v = thickness of flange delivering concentrated load
- T_r = thickness of flange of member to be stiffened
- A_s = area of flange delivering concentrated load.

The area of such stiffeners, A_s, shall be such that

\[ A_s \geq A_t - (T_r + 5k) \]

The ends of such stiffeners shall be fully but welded to the inside face of the flange adjacent to the concentrated tensile force. It shall be permissible to fit the stiffeners against the inside face of the flange adjacent to the concentrated compression force without welding. When the concentrated force is delivered by only one beam connected to an outside face of a strut, the length of the web stiffener shall extend for at least half the depth of the member, and the welding connecting it to the web shall be sufficient to develop a force of T_rA_s.

10.10.3 Plastic Hinges. Web stiffeners shall be provided at all plastic hinge points where the applied load exceeds 0.0064/F_v.

11. LOAD CAPACITIES OF CONNECTIONS.

The calculated load capacities of welds, bolts and rivets shall be taken as 1/0.60 times the values calculated from Section 9.
SECTION 11. FABRICATION AND ERECTION

11.1 GENERAL

11.1.1 Inspection. The Design Engineer and the Supervising Engineer (see Clause 1.4) or an appropriately qualified inspector representing them, shall have access at all reasonable times to all places where the work is being carried out and shall be provided by the Fabricator or the Contractor with all the necessary facilities for inspection while the work is in progress.

Note: A commentary on the requirements of this Clause may be found in SAA MA1.8, Fabrication, and MA1.9, Erection.

11.1.2 Supply. The requirements of AS 1227 shall apply to the supply of steel to the Fabricator.

11.1.3 Correction of Faults. Members which have been damaged during handling, and material or workmanship not complying with this standard, shall be made good by appropriate methods to the satisfaction of the Supervising Engineer.

11.1.4 Identification. Steel shall at all stages of fabrication be identifiable either by grade by an appropriate colour marking or other marking, or shall be classed as unidentifiable steel and only used in accordance with Clause 2.1.4.

Note: For identification of steel during fabrication, refer to commentary in SAA MA1.8, Section 8.7.1.

11.2 TOLERANCES.

11.2.1 General. A structural component shall satisfy the tolerance limits of this standard and of the contract documents. Unless otherwise specified, the tolerance on all structural dimensions shall be ± 2 mm.

Note: The Design Engineer should specify the tolerances on structural dimensions where either accurate location or temperature is critical.

11.2.2 Straightness. A structural member before erection shall not deviate from straightness or the specified configuration by more than the following:

- Struts . . . . . . . . . . . . L/1000
- Plates . . . . . . . . . . . . h/200
- Tubes . . . . . . . . . . . . L/600
- Other members . . . . . . . L/500

In this Clause, L is the length of the finished member, unless some lesser length is specified by the Design Engineer, and h is the lesser dimension of the web panel.

11.2.3 Length. The length of a member shall not deviate from the specified length by more than the following:

(a) A strut finished for full contact bearing . . . ± 2 mm.
(b) Any other member - 9000 mm long and under . . . ± 3 mm

11.2.4 Full Contact Splices. Full contact splices shall be produced by one of the three methods listed below:

11.2.4.1 Machine ends. If the ends of two butting lengths of a strut or the end of a strut and the contact face of an adjoining cap plate or baseplate are required by the Design Engineer to be in full contact (see Clause 6.5), such a requirement shall be deemed to be satisfied if the bearing surfaces are prepared so that when the butting strut length or lengths are aligned to within 1 in 1000 of their component length the maximum clearance between the abutting surfaces shall not exceed 1 mm.

11.2.4.2 Grounded ends. A bearing plate butted against a concrete or masonry component shall be deemed to be in full contact with that component if grout is placed between the bearing face and the butting member.

11.2.4.3 Butt-welded ends. A butt-welded splice shall be deemed to produce a full contact splice.

11.2.5 Struts Not Prepared for Full Contact. Clearance between the ends of successive struts shall not exceed 6 mm.

11.3 FABRICATION PROCEDURES.

11.3.1 General. Fabrication shall be carried out in accordance with this standard and with the requirements of the Design Engineer.

All material, before being assembled, shall be straightened or formed to the specified configuration by methods that will not reduce the properties of the material below the values assumed in the design.

11.3.2 Cutting. The cut edges of a member shall be free of gouges, burns and other defects which would adversely affect the serviceability of the member.

If flame cutting is used to form a tension member from a steel with a yield stress greater than 280 MPa, the cut edges shall be produced by combination of cutting conditions which yield a surface having properties suitable for the intended service.

In all flame cutting, the cut surface shall have a roughness not greater than the values given in Table 11.3.2.

Notes:
1. Roughness values may be estimated by comparison with surface replicates, such as the AWRA Flame Cut Surface replicates. For this replicate, the roughness values are 6.3 μm for Class 2 and 19 μm for Class 3.
2. Suitable techniques of flame cutting as recommended in AWRA Technical Note 5, ‘Flame Cutting of Steel’.

Roughness exceeding these values and occasional notches or gouges not more than 5 mm deep, on otherwise satisfactory surfaces, shall be removed by machining or grinding.

A re-entrant corner shall be shaped notch-free to a radius of at least 10 mm unless otherwise specified by the Design Engineer.

11.3.3 Welding.

11.3.3.1 General. Welding shall comply with AS 1554, where appropriate, and the additional requirements of this standard. Before any welding is carried out on an existing structure, the properties of the steel shall be determined from reliable documents or by precise tests.
TABLE 11.3.2
MAXIMUM CUT SURFACE ROUGHNESS

<table>
<thead>
<tr>
<th>Application</th>
<th>Maximum roughness (CLA) µm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal applications, i.e. where the face and edges remain intact or with minor dressing in die pockets</td>
<td>25.0</td>
</tr>
<tr>
<td>Tamper applications</td>
<td></td>
</tr>
<tr>
<td>Stress categories A, B, C</td>
<td>12.5</td>
</tr>
<tr>
<td>Others: forgings, etc.</td>
<td>2.5</td>
</tr>
</tbody>
</table>

Note: Where cold forming or risk of brittle fracture exists, special precautions should be taken. Further information may be found in AWRA Technical Note No 5, "Flame Cutting of Steels”.

11.3.3.2 Electrodes. Electrodes shall be selected so that the resulting weldment is compatible with the design requirements with respect to both strength and ductility and the relevant requirements of AS 1554.

11.3.4 Holes for Bolts and Rivets.

11.3.4.1 Sizes. The diameter of a completed hole shall be 2 mm larger than the nominal diameter of the bolt or rivet, unless otherwise specified by the Design Engineer. However, for high strength bolts, oversize and slotted holes in accordance with AS 1511 shall be permitted.

11.3.4.2 Alignment. All matching holes shall register with each other so that a gauge or drill 2 mm less in diameter than the holes shall pass freely through the assembled contact faces at right angles to them.

11.3.4.3 Finishing. Burrs, fins and other defects shall be removed from —
   (a) parts drilled separately;
   (b) parts subjected to fatigue loading or in which brittle fracture is considered possible as a result of such defects;
   (c) exposed surfaces;
   (d) fully punched holes.

Drifting to align holes shall be done in a manner that will not distort the metal or enlarge the holes.

11.3.4.4 Punching. Punching for holes for high strength bolts shall be in accordance with AS 1511.

Holes for plain carbon steel bolts may be punched full size in material up to 3600 kg/m² mm thick. Material of any thickness may be punched at least 3 mm undersize and then reamed.

11.3.4.5 Flame cutting. Flame cutting of a bolt or rivet hole shall not be permitted except in the case of foundation bolts in slab bases.

11.3.5 Bolting.

11.3.5.1 High-strength bolts. High-strength bolting shall comply with AS 1511.

11.3.5.2 Other steel bolts. Suitable tapered washers shall be positioned under the head or nuts of a bolt bearing on a bevelled surface.

The threaded portion of a bolt shall project through both ends of the nut by at least one thread after being tightened. The joined parts shall be firmly drawn together.

11.3.6 Riveting. The length of each rivet shall be such that when driven, a head of standard dimensions is formed.

A riveted member shall have all parts firmly drawn together and held in position before and during riveting. A service bolt shall be provided in every third or fourth hole in multiple-riveted connections.

A rivet shall be free from any warping or twist after heating, and when placed in position for driving shall be at red heat throughout and shall be upset to fill the hole completely and the closing head shall be fully formed into shape. A head of a countersunk rivet shall fill the countersink.

A burned rivet shall not be used and a driven rivet shall not show any cracks or pits, and where struck on the head with a rivet testing hammer shall be free from movement and vibration.

A loose rivet or a rivet with a head which is eccentric to the shank, or which is badly formed or deficient, shall be cut out and replaced before the structure is loaded.

11.3.7 Flattening Ends of Circular Hollow Sections. Where required by the design, the ends of a circular hollow section shall be formed or flattened provided that the method employed does not reduce the material properties below those assumed in the design.

11.3.8 Plated Joints. Holes for pinned joints shall be bored after welding and other fabrication procedures are complete. Pins shall be machined and fitted so that forces are distributed evenly to the adjoining members.

11.3.9 Surface Preparation. A steel face which is to be in contact with concrete or other steel-work shall be cleaned to remove all loose millscale, loose rust, oil, grease, dirt, globules of weld metal, weld slag and other foreign matter. Steel-to-steel surfaces shall be dry when contact is made. Steel-to-concrete interfaces shall be free from paint.

11.4 ERECTION.

11.4.1 Equipment Support. Equipment supported on partly erected steelwork shall not induce stresses in the steel greater than those permitted in this standard.

11.4.2 Setting Out Tolerances.

11.4.2.1 Level and alignment of beams. In erecting a structure, a beam shall be deemed to be correctly positioned when all connections including welded splices are completed, the beam complies with (a), (b) and (c) below and any bows in the beams in any plane shall be within 1/500 (see Clause 11.2.2): (a) A beam up to 2000 mm deep is within ± 7 mm of its correct level at connections to other members;
   (b) A beam over 2000 mm deep is within ± 10 mm of its correct level at connections to other members;
   (c) A web of a beam is within ± 3 mm of its correct position at connections to other members.
11.4.2.2 Alignment and plumbing of struts. The alignment and plumbing of struts shall be in accordance with the following requirements:

(a) The deviation of any point above the base of the strut from its correct position shall not exceed:
   (i) for a point up to 50 m above the base of the strut . . . 25 mm;
   (ii) for a point more than 50 m above the base of the strut . . . 25 mm and an additional 1 mm for every 3 m in excess of 50 m up to a maximum of 50 mm.

(b) If full contact is specified, the requirements of Clause 11.2.4 shall be satisfied, unless the Design Engineer permits the use of shims to reduce the measurable gaps to acceptable levels.

11.4.3 Safety During Erection. During erection of a structure, steelwork shall be made safe against erection stresses and loading conditions, including those due to erection equipment or its operation, and wind. Permanent connections shall not be made until correct alignment and camber, if any, has been obtained in those parts of the structure as will be stiffened thereby. For a multi-storey structure the above procedure shall be carried out progressively following as closely as is practicable behind the erection of each level.

Additional members used to facilitate erection shall be approved by the Design Engineer and affixed in a manner which does not weaken permanent steelwork.

11.4.4 Grouting at Supports.

11.4.4.1 Strut base and beams. Bedding under a strut base and bearing of a beam on masonry and concrete shall be carried out with portland cement grout or mortar.

For a multi-storey structure, this operation shall not be carried out until a sufficient number of bottom lengths of struts have been aligned, levelled and plumbed and are adequately braced by other structural components which have been levelled and are securely held by their permanent fastenings. Similar procedures shall be carried out in the erection of a single-storey structure. Steel packing or leveling nuts on the foundation bolts shall be under the baseplate to support the steelwork. The space under the steel shall be thoroughly cleaned and be free from excessive moisture immediately before grouting.

11.4.4.2 Bedding of grilles on concrete. The space under grillage beams and where specified, the space between two layers of grillage beams, shall be filled with the grout or mortar specified.

A strut baseplate shall bear directly on the grillage beams and where possible, grouting of grilles shall be carried out after the first lengths of struts have been erected, plumbed, aligned and levelled.

11.4.4.3 Grouting. Grout shall be of adequate strength, shall completely fill the space to be grouted and shall either be placed under pressure or placed by ramming against fixed supports.
APPENDIX A

DEFLECTION
(See Clause 3.3.5)

A1 NOTES ON DEFLECTION.
A1.1 General. The deformations of a structure and of its component members should be appropriate to the location, loading and function of the structure and the component members.
A1.2 Estimation. Deformations should be estimated using methods of analysis based on assumptions which reflect with reasonable accuracy the actual response of the structure to load. Full-scale or model tests may also be used to estimate deflections (see Clause 3.2.5).
A1.3 Special Conditions. In addition to the normal considerations which limit dead load deflection, both for aesthetic reasons and to aim at preventing cracking of walls and other elements, consideration should be given to any special conditions which may apply. Some of these are:
(a) Moment increase due to deflection. There is a possibility that a structure may become unstable if a combination of high axial load and deflection in a transverse direction can give rise to high secondary bending moments. If the lateral deflection of a building is not restricted for other reasons this effect should be considered.
(b) Load increase due to beam deflection. Overloading, increase in deflection, and possible instability may occur in beams as a consequence of the ponding of water on roofs of low slope.
(c) Damage to cladding. Relatively large lateral deflections may be acceptable in single-storey industrial structures since the stresses caused by secondary moments are usually small (see (a) above). However, such deflections may damage roof cladding, particularly in the vicinity of stiff end walls. Similar damage can occur to wall cladding.
(d) Resonance. Live loads applied rhythmically to a structure may cause resonance and hence cause deflections and stresses greater than would result if the maximum value of the load were applied statically. This may occur when a frequency component of the applied load is close to a natural frequency of the structure.
Live loads which are conducive to resonance include dancing, vehicular traffic, earthquake disturbance, unbalanced rotating machinery, and wind loads which result from the periodic shedding of vortices as can occur from tower-like structures and from roofs. In addition to general structural vibration, associated phenomena are ‘springiness’ in floors and the ‘flutter’ of roofs. A possible consequence of a repetitive load is metal fatigue.
A1.4 Conclusion. It is not considered practicable in this standard to give limiting values for every deflection case, as each must be considered on merit. However, for guidance only, some values which have been found to give satisfactory service in areas not subject to earthquakes, are given below for certain structures and components.

A2 DEFLECTION LIMITS FOR SPECIFIC CASES.
A2.1 Beams. For a beam supporting a concrete floor intended for human accommodation and having a span not exceeding 9 m, the calculated deflection due to live load alone and based on the unused section should not exceed —
Span

<table>
<thead>
<tr>
<th>Cantilever Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>180</td>
</tr>
</tbody>
</table>

A2.2 Purlins, Girts, Secondary Members. For a purlin, a girt, or a secondary member supporting asbestos-cement or metal sheathing and with a span not exceeding 6 m, the calculated deflection due to live load should not exceed —
Span

<table>
<thead>
<tr>
<th>Cantilever Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
</tr>
</tbody>
</table>

A2.3 Industrial Buildings. The horizontal deflection at eaves level of the internal frames of an industrial building, relative to the deflection in the same direction of the end wall at that level, should be appropriate to the capacity of the roof sheathing to accommodate the resulting shear distortion. Where no special provision is made for the sheathing to accommodate or resist this movement, the calculated eave movement due to live and wind load should be limited to —
Frame Spacing in End Bay

| 250                |

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APPENDIX B

FATIGUE
(See Clause 3.3.6)

B1 GENERAL. Fatigue cracks are usually initiated at points of high stress concentration. These stress concentrations may be caused by or associated with bolt or rivet holes, welds (including stray arc fusion), defects in materials, and local and general changes in geometry of members. The cracks usually propagate if loading is continued.

All details of members subjected to fatigue loading shall be designed so that stress concentrations will be as small as is practicable.

Where there is such loading, sudden changes of shape of a member or part of a member, especially in regions of tensile stress or local secondary bending, shall be avoided. Suitable steps shall be taken to avoid vibrations due to aerodynamic and like causes.

B2 LOADS AND STRESS CONCENTRATIONS. Where necessary, permissible stresses shall be reduced as described below to allow for the effects of fatigue. Allowance for fatigue shall be made for combinations of stresses due to dead load, live load and impact. Stresses due to wind and earthquakes may be ignored when fatigue is being considered.

Each element of the structure shall be designed for the number of stress cycles of each magnitude to which it is estimated that that element is liable to be subjected during the expected life of the structure. The number of cycles of each magnitude shall be estimated by the Design Engineer in the light of available data regarding the probable frequency of occurrence of each type of loading.

B3 LOADING CONDITIONS AND TYPE AND LOCATION OF MATERIAL.
Loading conditions shall be classified as shown in Table B1, and the type and location of material shall be categorised as shown in Table B2.

<table>
<thead>
<tr>
<th>Loading condition</th>
<th>Number of loading cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>From</td>
</tr>
<tr>
<td>1</td>
<td>20,000*</td>
</tr>
<tr>
<td>2</td>
<td>100,000</td>
</tr>
<tr>
<td>3</td>
<td>500,000</td>
</tr>
<tr>
<td>4</td>
<td>Over 5,000,000</td>
</tr>
</tbody>
</table>

* Approximately equivalent to two applications every day for 25 years.
† Approximately equivalent to ten applications every day for 25 years.
‡ Approximately equivalent to fifty applications every day for 25 years.
§ Approximately equivalent to two hundred applications every day for 25 years.

B4 MAXIMUM PERMISSIBLE STRESSES. The maximum stress shall not exceed the maximum permissible stress given elsewhere in this standard, and the maximum range of stress shall not exceed that given in Table B3.

Note: If a material is used whose thickness is less than that for which a minimum yield stress is specified, the value of \( F_y \) used in design should be that pertaining to the thinnest material to the same specification for which a minimum yield stress is specified.

B5 RIVETED AND BOLTED CONNECTIONS.

B5.1 Connections Made with Bolts Complying with AS 1252 and Subject to Tensile Fatigue Loading. For bolts complying with AS 1252 and installed to the requirements of AS 1511 which are subject to tensile fatigue loading, an allowance for fatigue shall not normally be required in calculations for the required number of
bolts in a bolted connection, except that the maximum stress shall not exceed the values given in AS 1511.

Notes:
1. Connections subject to more than 20000 cycles, or not more than 50000 cycles, of direct tension may be designed for the stress produced by the sum of applied and prying loads if the prying force does not exceed 10 percent of the externally applied load. If the prying force exceeds 10 percent, the allowable tensile stress given in AS 1511 should be reduced 40 percent, applicable to the external load alone.
2. Connections subject to more than 50000 cycles of direct tension may be designed for the stress produced by the sum of applied and prying loads if the prying load does not exceed 5 percent of the externally applied load. If the prying force exceeds 5 percent, the allowable tensile stress given in AS 1511 should be reduced 50 percent, applicable to the external load alone.

B5.2 Other Mechanical Fasteners Subject to Tensile Fatigue Loading. The use of bolts not complying with AS 1252 or not installed to the requirements of AS 1511, or rivets, is not recommended where such fasteners are subjected to tensile fatigue loading.

B5.3 Rivets, Bolts and Threaded Parts Subjected to Cyclic Loading in Shear. Rivets, bolts, and threaded parts subjected to cyclic loading in shear may be designed for the permissible stresses given in Table 9.5.2 or AS 1511, as appropriate, to the extent of the fatigue strength of the fasteners themselves is concerned.
<table>
<thead>
<tr>
<th>General condition</th>
<th>Situation</th>
<th>Kind of stress*</th>
<th>Stress category (see Table B2)</th>
<th>Illustrative example No. (see Fig. B1)?</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain material</td>
<td>Base metal with rolled or cleaned surfaces</td>
<td>T or Rev.</td>
<td>A</td>
<td>1, 2</td>
</tr>
<tr>
<td>Built-up members</td>
<td>Base metal and weld metal in members, with or without attachments, out-of-plane or transverse stiffeners, connected by continuous complete penetration or incomplete penetration butt welds or continuous fillet welds parallel to the direction of applied stress</td>
<td>T or Rev.</td>
<td>B</td>
<td>3, 4, 5, 6</td>
</tr>
<tr>
<td></td>
<td>Calculated stress, ( f_n ), in base metal at toe of welds or grider plates or fillets adjacent to welded transverse stiffeners</td>
<td>T or Rev.</td>
<td>C</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>Base metal at end of partial-length welded cover plates having square or tapered ends, with or without welds across the ends</td>
<td>T or Rev.</td>
<td>E</td>
<td>5</td>
</tr>
<tr>
<td>Mechanically fastened connections</td>
<td>Base metal at gross section of high-strength bolted-plate-type connections complying with AS 1311, with fasteners on tension side of member and flange plate on compression side, which induce out-of-plane bending in connection material</td>
<td>T or Rev.</td>
<td>B</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>Base metal at net section of high-strength bolted bearing-type connections complying with AS 1311</td>
<td>T or Rev.</td>
<td>B</td>
<td>8, 9</td>
</tr>
<tr>
<td></td>
<td>Base metal at net section of other mechanically fastened joints</td>
<td>T or Rev.</td>
<td>D</td>
<td>8, 9</td>
</tr>
<tr>
<td>Fillet welded connections</td>
<td>Base metal at intermittent fillet welds</td>
<td>T or Rev.</td>
<td>E</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Base metal at junction of axially loaded members, with fillet welds and connections, welds shall be disposed about the axis of the member so as to balance weld stresses</td>
<td>T or Rev.</td>
<td>E</td>
<td>17, 18, 20, 28 and 29</td>
</tr>
<tr>
<td></td>
<td>Weld metal of continuous or intermittent longitudinal or transverse fillet welds</td>
<td>S or F</td>
<td>5, 17, 18, 25, 28</td>
<td></td>
</tr>
<tr>
<td>Butt welded</td>
<td>Base metal and weld metal at complete penetration butt welded splice of parts of similar cross section ground flush, with grinding in the direction of applied stress and weld soundness established by radiographic or ultrasonic inspection in accordance with AS 1354</td>
<td>T or Rev.</td>
<td>B</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>Base metal and weld metal at complete penetration butt welded splice at transitions in width or thickness, with welds ground to provide slopes no greater than 1 to 2.5 with grinding in the direction of applied stress, and weld soundness established by radiographic or ultrasonic inspection in accordance with AS 1354</td>
<td>T or Rev.</td>
<td>B</td>
<td>12, 13</td>
</tr>
<tr>
<td></td>
<td>Base metal and weld metal at complete penetration butt welded splice, with or without transitions having slopes no greater than 1 to 2.5 when encasement is not required and weld soundness is not established by radiographic or ultrasonic inspection in accordance with AS 1354</td>
<td>T or Rev.</td>
<td>C</td>
<td>10, 11, 12, 13</td>
</tr>
<tr>
<td></td>
<td>Weld metal on incomplete penetration transverse butt welds, based on effective throat area of the weld or welds</td>
<td>T or Rev.</td>
<td>F</td>
<td>16</td>
</tr>
<tr>
<td>Plug or slot welded</td>
<td>Base metal at plug or slot welds</td>
<td>T or Rev.</td>
<td>E</td>
<td>27</td>
</tr>
<tr>
<td></td>
<td>Shear on plug or slot welds</td>
<td>S or F</td>
<td>F</td>
<td>27</td>
</tr>
</tbody>
</table>
### TABLE B2 (continued)

<table>
<thead>
<tr>
<th>General condition</th>
<th>Kind of stress*</th>
<th>Stress category (see Table B2)</th>
<th>Illustrative example (see Fig. B2)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Attachments</td>
<td>Base metal at detail of any length attached by butt welds subject to transverse and/or longitudinal loading, when the detail exhibits a transition radius ( R ) of 50 mm or greater, with the weld penetration ground smooth.</td>
<td>T or Rev.</td>
<td>B 14</td>
</tr>
<tr>
<td></td>
<td>( 600 &gt; R \geq 100 ) mm</td>
<td>T or Rev.</td>
<td>C 14</td>
</tr>
<tr>
<td></td>
<td>( 150 &gt; R \geq 50 ) mm</td>
<td>T or Rev.</td>
<td>D 14</td>
</tr>
<tr>
<td></td>
<td>Base metal at detail attached by butt welds or fillet welds subject to longitudinal loading, with transition radius, ( R ), less than 50 mm.</td>
<td>T or Rev.</td>
<td>B 15, 25, 26</td>
</tr>
<tr>
<td></td>
<td>( 50 \text{ mm} &lt; a \leq 12b \text{ or } 100 \text{ mm} )</td>
<td>T or Rev.</td>
<td>E 15, 25, 26</td>
</tr>
<tr>
<td></td>
<td>( a &gt; 12b \text{ or } 100 \text{ mm} )</td>
<td>T or Rev.</td>
<td>E 25, 26</td>
</tr>
<tr>
<td></td>
<td>where ( a ) = detail dimension parallel to the direction of stress ( b ) = detail dimension normal to the direction of stress and the surface of the base metal.</td>
<td>T or Rev.</td>
<td></td>
</tr>
<tr>
<td>Base metal at detail of any length attached by butt welds or incomplete penetration butt welds and welds in the direction parallel to the stress, when the detail exhibits a transition radius ( R ), less than 50 mm, with weld termination ground smooth.</td>
<td>T or Rev.</td>
<td>B 19</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( 600 &gt; R \geq 100 ) mm</td>
<td>T or Rev.</td>
<td>C 19</td>
</tr>
<tr>
<td></td>
<td>( 150 &gt; R \geq 50 ) mm</td>
<td>T or Rev.</td>
<td>D 19</td>
</tr>
<tr>
<td>Base metal at detail attached by butt welds or fillet welds, where the detail dimension parallel to the direction of stress, ( a ), is less than 50 mm</td>
<td>T or Rev.</td>
<td>C 25, 24, 25</td>
<td></td>
</tr>
<tr>
<td>Base metal at a stud-type shear connector attached by fillet weld</td>
<td>T or Rev.</td>
<td>C 22</td>
<td></td>
</tr>
<tr>
<td>Shear stress on nominal area of stud-type shear connectors</td>
<td>S</td>
<td>F 22</td>
<td></td>
</tr>
</tbody>
</table>

* "T" signifies range in tensile stress only;
  "Rev" signifies a range involving reversal of tensile or compressive stress;
  "S" signifies range in shear including shear stress reversal.

† These examples are provided as guidelines and are not intended to exclude other reasonably similar situations.

### TABLE B3

**MAXIMUM PERMISSIBLE STRESS RANGE**

<table>
<thead>
<tr>
<th>Category (Stress Table B2)</th>
<th>Maximum permissible stress range</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Loading condition 1</td>
</tr>
<tr>
<td></td>
<td>( P_{ux} )</td>
</tr>
<tr>
<td>A</td>
<td>165</td>
</tr>
<tr>
<td>B</td>
<td>150</td>
</tr>
<tr>
<td>C</td>
<td>150</td>
</tr>
<tr>
<td>D</td>
<td>150</td>
</tr>
<tr>
<td>E</td>
<td>150</td>
</tr>
<tr>
<td>F</td>
<td>150</td>
</tr>
</tbody>
</table>

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Fig. B1. ILLUSTRATIVE EXAMPLES OF STRESS SITUATIONS
Fig. 81. ILLUSTRATIVE EXAMPLES OF STRESS SITUATIONS (continued)
### Appendix C

#### Minimum Yield Stresses for Steel to AS 1163, AS 1204 and AS 1205

<table>
<thead>
<tr>
<th>Minimum yield stress (MPa)</th>
<th>Value of $f_y$ in tables (MPa)</th>
<th>Steel grade and specification</th>
<th>Form</th>
<th>Thickness of material (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>360</td>
<td>360</td>
<td>350 350 L 350 L 15</td>
<td>AS 1204</td>
<td>Up to 12</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>350</td>
<td></td>
<td>AS 1163</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>350 350 L 350 L 15</td>
<td>AS 1204</td>
<td>Up to 12</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>340</td>
<td>340</td>
<td>350 350 L 350 L 15</td>
<td>AS 1204</td>
<td>Up to 12</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>330</td>
<td>330</td>
<td>AS 1204</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>350 350 L 350 L 15</td>
<td>AS 1204</td>
<td>80 to 100</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>320</td>
<td></td>
<td>350 AS 1204</td>
<td></td>
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<tr>
<td>300</td>
<td></td>
<td>350 AS 1204</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>280</td>
<td>280</td>
<td>350 350 L 350 L 15</td>
<td>AS 1204</td>
<td>Up to 8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>260</td>
<td>260</td>
<td>350 350 L 350 L 15</td>
<td>AS 1204</td>
<td>Up to 12</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>250</td>
<td></td>
<td>350 AS 1204</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(continued)
<table>
<thead>
<tr>
<th>Minimum yield stress (MPa)</th>
<th>Value of $F_y$ (MPa)</th>
<th>Steel grade and specification</th>
<th>Form</th>
<th>Thickness of material (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>240</td>
<td>240</td>
<td>250 AS 1304</td>
<td>Plate, strip and floor plate</td>
<td>20 to 80</td>
</tr>
<tr>
<td></td>
<td></td>
<td>250 AS 1304</td>
<td></td>
<td>40 to 180</td>
</tr>
<tr>
<td>210</td>
<td>230</td>
<td>250 AS 1304</td>
<td>Plate, strip and floor plate</td>
<td>80 to 180</td>
</tr>
<tr>
<td></td>
<td></td>
<td>250 AS 1304</td>
<td>Sections and flat bars</td>
<td>Over 40</td>
</tr>
<tr>
<td></td>
<td></td>
<td>250 AS 1304</td>
<td>Round, hexagonal and square bars</td>
<td>50 and over</td>
</tr>
<tr>
<td>200</td>
<td></td>
<td>200 AS 1304</td>
<td>Plate, strip and floor plate</td>
<td>Up to 12</td>
</tr>
<tr>
<td></td>
<td></td>
<td>200 AS 1304</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C200 AS 1163</td>
<td>Hollow sections</td>
<td>All</td>
</tr>
</tbody>
</table>
APPENDIX D

LIST OF REFERENCES ON THE ELASTIC FLEXURAL-TORSIONAL BUCKLING OF STEEL BEAMS

Textbooks:


List of references prior to 1961:


Values of elastic critical loads and effective length factors:

E1 EFFECTIVE LENGTH OF STRUTS IN RECTANGULAR FRAMES.
(See Clause 6.3)

E1.1 General. The principle that a strut has an effective length is used to relate the in-plane elastic buckling load of a framework as a whole to that of an equivalent pinned-end strut for which the allowable axial stresses have been given in Clause 4.1. An accurate frame buckling analysis for all but the simplest of framed structures would involve considerable calculation and the results would be valid only for the loading pattern considered. Nevertheless, the accurate estimation of the effective length is important for heavily loaded struts, particularly if proportioned in high strength steel.

E1.2 Sideways Prevented. For a strut in a rigid-jointed frame where sideways is effectively limited as in Clause 6.3.2, the effective length shall satisfy formula E1:

\[
\frac{G_s d_a}{4} + \frac{G_s}{G_a + G_b} \left( 1 - a \cot a \right) + \frac{2 \tan(a/2)}{a} = 1
\]

E1

In this formula, the stiffness ratios \( G_s \) and \( G_a \) for each end of the strut may be calculated from:

\[
G = \frac{f_{lc} l_{lc}}{\Sigma \left[ \frac{f_a}{l_a} \times y \right]}
\]

where

\[
\gamma = \frac{b}{\tan b} = \frac{b}{\sin b} \times \frac{l}{r}
\]

NOTE: For members with zero axial load, \( f_{lc} = 0 \), \( b = 0 \), and \( \gamma = 1 \).

The summations of stiffness are carried out for all members other than the strut being designed which are rigidly connected at each joint and which lie in the appropriate plane of buckling of the strut.

As an alternative for regular rectangular frames (see Paragraph E1.5), the stiffness ratios \( G_s \) and \( G_a \) may be calculated from:

\[
G = \frac{\sum f_a l_a}{\sum f_a l_a} = 1
\]

E3

in which the summations of stiffness are carried out for all members (including that part of the strut under consideration and the part above or below) which are rigidly connected at each joint and which lie in the appropriate plane of buckling of the strut.

The value of the effective length/actual length ratio \( (l/l) \) which satisfies formula E1 can be found directly once \( G_s \) and \( G_a \) have been calculated by using the chart in Fig. E1.

E1.3 Sideways Not Prevented. For a strut in a regular rectangular frame which depends on flexural action to limit sway as in Clause 6.3.3, the effective length shall satisfy formula E4.

\[
\frac{G_s d_a}{4} + \frac{a \cot a}{G_a + G_b} = 1
\]

E4

In this formula, the stiffness ratios \( G_s \) and \( G_a \) for each end of the strut may only be calculated from formula E3, unless an alternative formula is calculated from a rational elastic buckling analysis.

The value of the effective length/actual length ratio \( (l/l) \) which satisfies formula E4 can be found directly once \( G_s \) and \( G_a \) have been calculated by using the chart in Fig. E2.

E1.4 Notation. In formulas E1, E2, E3, and E4:

\[
a = \frac{l_{lc} f_{lc}}{l}, \quad l_{lc} = \text{actual calculated stress}
\]
$I_a = \text{second moment of area of a restraining beam, neglecting any casing, about the axis normal to the buckling plane}$

$E = \text{second moment of area of a strut, neglecting any casing, about the axis normal to the buckling plane}$

$I_{de} = \text{second moment of area of the strut being designed, neglecting any casing, about the axis normal to the buckling plane}$

$I_{e} = \text{second moment of area of the member other than the strut being designed, neglecting any casing, about the axis normal to the buckling plane}$

$L_a = \text{actual unbraced length of a restraining beam}$

$L_e = \text{actual length of a strut}$

$L_{de} = \text{actual length of the strut being designed}$

$L_e = \text{actual unbraced length of any member other than the strut being designed}$

**E1.5 Application.**

**E1.5.1 Assumptions.** Formulas E1 and E4 have been based upon a rational elastic stability analysis of an elastically restrained member of uniform section loaded in compression. In applying the analysis to members of a regular rectangular framework the following assumptions have been made:

(a) The effect on the frame critical load of primary bending moments caused by transverse loading of beams is neglected.

(b) Frame buckling occurs in the plane under consideration and the restraining moment at a joint provided by the beams is distributed between the upper and lower strut lengths in proportion to their E.I. ratios.

(c) All struts in the frame are critically loaded, with the beams providing the least possible rotational restraint at a joint consistent with the appropriate assumption as to whether sway can or cannot occur.

**E1.5.2 Use of charts.** In using the charts to determine the effective length of a strut, consideration should be given to the following factors:

(a) Where a beam is connected to a strut by a detail with obviously negligible moment transmitting capacity, the contribution of that beam to the appropriate G term is zero.

(b) At a strut base which is not rigidly connected to a footing, G is theoretically infinite but should be taken as 10 for a practical design. If a strut end is rigidly attached to a properly designed footing, G is theoretically zero but should be taken as 1.0 unless analysis would justify smaller values.

(c) Modifications to the beam stiffness ratio ($L/E$ or $I/I_a$) should be made if the condition at the far end of the beam is other than that assumed in the above analysis, namely, a rigid connection to another strut critically loaded.

With sway effectively restrained by bracing or equivalent, the beam stiffness ratio should be increased by multiplying by the following factors:

- For far end of the beam hinged: $1.5$
- For far end effectively encased: $2.0$
- For an unbraced frame, the beam stiffness ratio must be increased by multiplying by the following factors:
  - For far end effectively encased: $0.67$
  - For far end of the beam hinged: $0.5$

**E2 EFFECTIVE LENGTH OF STRUTS IN TRIANGULATED FRAMES.** The optimum design for a truss may be that for which all members buckle or yield at the same load level on the truss and hence for which no restraint would be applied to compression members by the joints. The effective length, $L$, of a compression member would then be equal to the full distance between braced panel points, $L$. For a triangulated frame where an optimum design has not been achieved, the effective length $L$ of a compression member may be determined by a rational elastic buckling analysis, or by using formulas E1 and E2. Alternatively, in triangulated frames of a steel whose yield stress is 280 MPa or less and whose members adjacent to the strut under consideration are stressed to less than 0.75 times their maximum permissible capacities, the effective length of a strut shall be not less than 0.85L. For a battened strut or for a discontinuous strut connected at each end by a single bolt or rivet, or a gusset having no significant stiffness in the plane of buckling, the effective length of the strut should not be less than the actual length.

Note: The Standards Association of Australia is not prepared to make recommendations at this stage concerning the effective length of unbraced compression chords in trusses where the ends of the trusses are not prevented from rotating.

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Fig. E1. SWAY PREVENTED

Fig. E2. SWAY NOT PREVENTED
INDEX

Clause No

Acceptance of structural steels
Allowable stresses—see Stresses, maximum permissible
Angles—
as struts—
internal
two components back-to-back
as tension members—
encastre connection
two components back-to-back
unsupported laterally
Bolt plates for struts—
general
grillage beams, bearing on
grafting under
stiffeners, machining
buttressed struts—see under Struts, composite beams
backing of—
deformation
maximum permissible stress
reference
calculation of bending stresses by equivalent properties
critical flange—see under Flanges, beam, critical
deflection
design of effective length—
general
restraints
effective span—
flange—see under Flanges
grillage beams, bearing on
lateral restraining systems
maximum permissible stresses—see under Struts, maximum permissible plastic design
moment capacity of a beam, maximum—of members subjected to combined bending and axial force
shear capacity, maximum—subjected to axial force
setting out tolerances
tolerance ratio, maximum
supports, bearing under
struts—see under Wells
web—see under Wells
web thicknesses—see under Stiffeners, web
Bearing stresses—see under Stresses, maximum permissible
Bending stresses—see under Stresses, maximum permissible, beams
Bolts, general—
connection design—
choice of fasteners
combined connections
combined stresses
minimum design force
maximum force, plastic design
stresses
edge distances
effective cross-sectional area
fabrication procedures
holes
locking of nuts
maximum pitch
plain carbon steel, standard for plastic design
standards for plastic design
stresses
9.3 Table 9.5.2

Clause No

Bolts, high-strength—
end distance in bearing
fabrication and erection
holes
plastic design
standard for stresses
shallow creased
flange, effective geometrical properties of section
shear, average, in webs
web area, effective, reducing shear
Brittle fracture—
general
Building Authority—
acceptance of steel by
definition
Casting—
every cast design, approval for
new materials and methods, approval for
Butt welds—see under Wells
Cap plates for struts—
general
tolerances, machining
Cased beams
Cased struts—
axial compression and bending
permissible force, details
Casing—
standard for
Chordons—
as struts—
two components back-to-back
tension members—
excessively connected
two components back-to-back
Circular hollow sections—see under Hollow sections, circular
Columns—see under Struts
Combined bending and axial force—
combined stresses, axial and bending
plastic design
tension members
Combined connections, non-tension and other
Combined stresses—see also under Combined Bending and Axial Force, bolts and rivets, shear and tension
Composite struts—see under Struts
Compression members—see under Struts
Concrete
cased beams
cased struts
grillage beams
standard for
Connections—see also under names of fastener type, composite member type, etc.
plastic design
Corrosion protection—
filter welds, intermittent
general
pitch of fasteners, maximum
Critical flange—see under Flanges, beam, critical
definition—
administrative
general
technical
Existing terms
COPyRİGHT
<table>
<thead>
<tr>
<th>Clause No</th>
<th>Deflection limits—</th>
</tr>
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<tbody>
<tr>
<td>3.5.5</td>
<td>general guidelines</td>
</tr>
<tr>
<td>Appendix A</td>
<td>for the Engineer</td>
</tr>
<tr>
<td>3.4</td>
<td>general requirements</td>
</tr>
<tr>
<td>Section 3</td>
<td></td>
</tr>
<tr>
<td>3.3</td>
<td>Design considerations, other than Methods...</td>
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</tbody>
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<table>
<thead>
<tr>
<th>Clause No</th>
<th>Design methods—</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.2.5(a)</td>
<td>full-scale tests</td>
</tr>
<tr>
<td>3.2.5(b)</td>
<td>general model tests</td>
</tr>
<tr>
<td>3.2.5(b)</td>
<td>plastic design</td>
</tr>
<tr>
<td>(Method 1)</td>
<td></td>
</tr>
<tr>
<td>(Method 2)</td>
<td></td>
</tr>
<tr>
<td>3.2.3</td>
<td>general structure</td>
</tr>
<tr>
<td>3.2.4</td>
<td>simplified</td>
</tr>
<tr>
<td>3.2.2(a)</td>
<td>rectangular/framed struc.</td>
</tr>
<tr>
<td>3.2.2(b)</td>
<td>triangulated structures</td>
</tr>
<tr>
<td>3.14</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Clause No</th>
<th>Eccentricity in struts—</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.4.2</td>
<td>location of beam reaction</td>
</tr>
<tr>
<td>6.4.1</td>
<td>maximum moment</td>
</tr>
<tr>
<td>9.6.4</td>
<td>Effective cross-section—see under Geometrical properties</td>
</tr>
<tr>
<td>9.6.2</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Clause No</th>
<th>Effective length—</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.9</td>
<td>beams</td>
</tr>
<tr>
<td>10.7</td>
<td>plastically designed frames</td>
</tr>
<tr>
<td>4.6</td>
<td>slenderess ratio calculations</td>
</tr>
<tr>
<td>4.6</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Clause No</th>
<th>Fatigue—</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.4.1.2</td>
<td>complete-penetration butt welds, interna-</td>
</tr>
<tr>
<td>9.4.1.2</td>
<td>tional pressure</td>
</tr>
<tr>
<td>9.4.1.2</td>
<td>fillet welds, intermittent</td>
</tr>
<tr>
<td>3.3.6</td>
<td>general requirements</td>
</tr>
<tr>
<td>Appendix B</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Clause No</th>
<th>Faults, corrosion of—</th>
</tr>
</thead>
<tbody>
<tr>
<td>11.1.3</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Clause No</th>
<th>Flanges, general—</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.10.1</td>
<td>applying force to another member</td>
</tr>
<tr>
<td>10.10.2</td>
<td>lateral restraining systems</td>
</tr>
<tr>
<td>1.3.6</td>
<td>moment arms</td>
</tr>
<tr>
<td>3.3.2(b)</td>
<td>outrig-</td>
</tr>
<tr>
<td>8.1.1</td>
<td>plastic design</td>
</tr>
<tr>
<td>10.9</td>
<td>lateral restraint</td>
</tr>
<tr>
<td>10.8.1</td>
<td>unsupported widths</td>
</tr>
<tr>
<td>10.8.2</td>
<td>thickness</td>
</tr>
<tr>
<td>4.3.10.8</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Clause No</th>
<th>Flanges, beam—</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.1</td>
<td>beams with unequal flanges</td>
</tr>
<tr>
<td>5.2.2</td>
<td>intermediate lateral restraint for</td>
</tr>
<tr>
<td>5.5.6</td>
<td>load on flange connection laterally</td>
</tr>
<tr>
<td>5.13.2.4</td>
<td>stiffeners, load bearing, for nominal</td>
</tr>
<tr>
<td>5.9.3</td>
<td>with intermediate lateral restraint</td>
</tr>
<tr>
<td>5.9.3</td>
<td>without intermediate lateral restraint</td>
</tr>
<tr>
<td>5.12.2</td>
<td>curtailment of flange plates</td>
</tr>
<tr>
<td>5.3.1</td>
<td>design details</td>
</tr>
<tr>
<td>5.5</td>
<td>removal of part of flange at end</td>
</tr>
<tr>
<td>5.12.1</td>
<td>web connections, plate girders</td>
</tr>
<tr>
<td>5.12.3</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Clause No</th>
<th>Flanges, deductions for effective section—</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.3.1</td>
<td>outside diameters</td>
</tr>
<tr>
<td>4.3.2</td>
<td>unsupported widths in box sections</td>
</tr>
<tr>
<td>4.3.3</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Clause No</th>
<th>Fracture, brittle—</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.5.6</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Clause No</th>
<th>Fully rigid design method—see under Design methods—</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Clause No</th>
<th>Geometrical properties—</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.5</td>
<td>bolts, screw tension rods</td>
</tr>
<tr>
<td>4.5.1</td>
<td>circular hollow sections</td>
</tr>
<tr>
<td>4.5.1</td>
<td>flange outward</td>
</tr>
<tr>
<td>4.5.2</td>
<td>flange widths, box sections</td>
</tr>
<tr>
<td>5.18.4</td>
<td>for resisting shear</td>
</tr>
<tr>
<td>6.1(b)</td>
<td>general</td>
</tr>
<tr>
<td>6.4</td>
<td>holes, deductions for—</td>
</tr>
<tr>
<td>6.4</td>
<td>details</td>
</tr>
<tr>
<td>6.4</td>
<td>radius of gyration for calculating slenderess ratios</td>
</tr>
<tr>
<td>6.4</td>
<td>general</td>
</tr>
<tr>
<td>6.4</td>
<td>gross cross-section—</td>
</tr>
<tr>
<td>6.4</td>
<td>bolts and tension rods</td>
</tr>
<tr>
<td>6.4</td>
<td>general</td>
</tr>
<tr>
<td>6.4</td>
<td>slenderess ratio, maximum</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Clause No</th>
<th>Girths—</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.2.2</td>
<td>deflection—</td>
</tr>
<tr>
<td>6.2.2</td>
<td>general</td>
</tr>
<tr>
<td>6.4</td>
<td>Grillage beams—</td>
</tr>
<tr>
<td>6.4</td>
<td>concrete, bedding on—</td>
</tr>
<tr>
<td>6.4</td>
<td>general</td>
</tr>
<tr>
<td>6.4</td>
<td>grouting—</td>
</tr>
<tr>
<td>6.4</td>
<td>separators and diaphragms—</td>
</tr>
<tr>
<td>6.4</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Clause No</th>
<th>Groove cross-section—see under Geometrical properties—</th>
</tr>
</thead>
<tbody>
<tr>
<td>11.4.4</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Clause No</th>
<th>Holing sections—</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.4</td>
<td>details</td>
</tr>
<tr>
<td>4.4</td>
<td>general</td>
</tr>
<tr>
<td>6.4</td>
<td>hollow sections—</td>
</tr>
<tr>
<td>6.4</td>
<td>standard for—</td>
</tr>
<tr>
<td>5.4.6</td>
<td>stresses, maximum permissible for</td>
</tr>
<tr>
<td>5.4.6</td>
<td>yield stresses</td>
</tr>
<tr>
<td>Appendix C</td>
<td></td>
</tr>
<tr>
<td>Hollow sections, circular—see also under Hollow sections</td>
<td>5.10.4</td>
</tr>
<tr>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Effective cross-section resisting shear</td>
<td>11.1.7</td>
</tr>
<tr>
<td>Effective geometrical properties of section</td>
<td>4.3.3</td>
</tr>
<tr>
<td>Ends, butt, incomplete penetration</td>
<td>11.1.4</td>
</tr>
<tr>
<td>Hollow sections, rectangular—see under Hollow sections and Box sections</td>
<td>3.3.3</td>
</tr>
<tr>
<td>Horizontal shear due to twisting</td>
<td>3.3.3</td>
</tr>
<tr>
<td>Identification of steel</td>
<td>3.3.3</td>
</tr>
<tr>
<td>Industrial buildings, deflection</td>
<td>3.3.3</td>
</tr>
<tr>
<td>Inspection</td>
<td>3.3.3</td>
</tr>
<tr>
<td>Load struts—see under Strut, composite</td>
<td>3.3.3</td>
</tr>
<tr>
<td>Lateral force—</td>
<td>3.3.3</td>
</tr>
<tr>
<td>Lateral restraint—</td>
<td>3.3.3</td>
</tr>
<tr>
<td>Plastic design</td>
<td>3.3.3</td>
</tr>
<tr>
<td>Purlins and girts</td>
<td>3.3.3</td>
</tr>
<tr>
<td>Light gauge steel</td>
<td>3.3.3</td>
</tr>
<tr>
<td>Loads—</td>
<td>3.3.3</td>
</tr>
<tr>
<td>Fatigue—see under Fatigue</td>
<td>3.3.3</td>
</tr>
<tr>
<td>Fluctuations—see under Fatigue</td>
<td>3.3.3</td>
</tr>
<tr>
<td>Load factor—</td>
<td>3.3.3</td>
</tr>
<tr>
<td>Material—</td>
<td>3.3.3</td>
</tr>
<tr>
<td>Mould tests—see under Design methods, experimentally based</td>
<td>3.3.3</td>
</tr>
<tr>
<td>New materials or methods</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Notation—</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Overturning—see Stability</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Packing—</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Reinforcement, concrete—see under Concrete</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Restraint systems, lateral</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Rivets—</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Rolled sections</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Stress—see under Stress, maximum permissible</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Strength</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Stiffeners, horizontal</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Stiffeners, intermediate</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Stiffeners, vertical</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Stresses—see under Stress, maximum permissible</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Test methods</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Welds, butt, incomplete penetration</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Width</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Shear capacity, plastic design</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Shear strength—see under Stress, maximum permissible</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Shear stress—see under Stress, maximum permissible</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Simple design method—see under Design methods</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Strength—see under Stress, maximum permissible</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Structural members</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Strut, composite</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Subject to local buckling</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Tension</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Torsion</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Tubular sections</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Ultra high-strength steel</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Unit weight, steel</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Unit weight, steel—</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Use of sections</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Welded sections</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Welding, butt joint, complete penetration</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Welding, butt, incomplete penetration</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Welding, fillet</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Welding, partial penetration</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Welding, partial penetration—see under Welding</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Welding, root</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Welds, butt, incomplete penetration</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Welds, butt, in incomplete penetration</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Welds, complete penetration</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Welds, complete penetration—see under Weld</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Welds, fillet</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Welds, fillet—see under Welds</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Welds, partial penetration</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Welds, partial penetration—see under Welds</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Welds, root</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Welds, root—see under Welds</td>
<td>1.1.1</td>
</tr>
<tr>
<td>Welds, root</td>
<td>1.1.1</td>
</tr>
</tbody>
</table>

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NOTATION

The following notation comprises a reprint of the notation given in Clause 1.6:

\( A_s \) = the gross cross-sectional area of a concrete-encased strut neglecting any casing in excess of 25 mm from the overall dimensions of the steel section and neglecting any applied finish

\( A_e \) = the effective cross-sectional area of a steel member

\( A_s \) = the effective sectional area resisting shear for calculating the average shear stress or the maximum shear capacity in a member

\( b \) = the overall width of the flange of a section

\( b_s \) = the lesser clear dimension of a web panel

\( b_{x,y} \) = the width of a flange between two adjacent lines of connections to other parts of a member, or other line of support

\( D \) = the depth of a section measured parallel to the web

\( d \) = the clear depth of the web of a rolled I-section or channel between root fillets

\( d_s \) = for the web of a beam without horizontal stiffeners, the clear distance between the flanges, neglecting fillets, or the clear distance between the inner toes of the flange angles, as appropriate

\( d_{x,y} \) = for the web of a beam with horizontal stiffeners, the clear distance between the horizontal stiffener and the tension flange, neglecting fillets, or the inner toes of the tension flange angles, as appropriate

\( E \) = the modulus of elasticity for steel, taken as 2 × 10^6 MPa in this standard

\( F' \) = the minimum compressive strength of concrete as defined in AS 1480

\( F_{ee} \) = the maximum permissible compressive stress in an axially loaded strut not subjected to bending

\( F_{ee} \) = the maximum permissible tensile stress in an axially loaded tension member not subjected to bending

\( F_{pe} \) = the maximum permissible stress due to bending in a member not subjected to axial force

\( F_{pe} \) = the maximum permissible compressive stress due to bending in a member not subjected to axial force

\( F_{pe} \) = the maximum permissible tensile stress due to bending in a member not subjected to axial force

\( F_{se} \) = the elastic buckling stress of a strut

\( F_{se} \) = the maximum permissible stress in bearing in a member

\( F_{te} \) = the tensile strength pertaining to the steel used in a member

\( F_{ae} \) = the maximum permissible average stress in shear in a member

\( F_{pe} \) = the maximum permissible stress in shear in a member

\( F_v \) = the yield stress pertaining to the steel used in a member; where \( F_v \) is used in a dimensionally inconsistent expression in this standard, its value shall be expressed in MPa (see Appendix C)

\( G_s \) = the calculated average stress in a member due to an axial compressive force

\( f_{se} \) = the calculated average stress in a member due to an axial tensile force

\( f_{te} \) = the calculated compressive stress in a member due to bending about a principal axis

\( f_{te} \) = the calculated tensile stress in a member due to bending about both principal axes

\( I \) = the second moment of area (moment of inertia) of the cross section of a pair of web stiffeners about the centroid of a beam web, or of a single stiffener about the face of a beam web, as appropriate

\( L \) = the span of a member

\( M_{pe} \) = the calculated maximum moment capacity of a beam

\( M_{pe} \) = the calculated maximum moment capacity of a member subjected to bending and axial load

\( P \) = an axial force, compressive or tensile, in a member or part of a member

\( P_{pe} \) = the calculated maximum load capacity of a strut

\( P_{pe} \) = the calculated maximum load capacity of a tension member

\( Q \) = a force applied transversely to a member

\( R \) = the reaction of a beam at a support

\( r \) = the radius of gyration of a section

\( S \) = the longitudinal spacing between the centres of successive vertical stiffeners in a beam

\( T \) = in a flanged section, the mean thickness of the flange, or the area of the flange divided by its width, or the thickness of the flange as given in AS 1113, as appropriate

\( T_{pe} \) = the thickness of the flange of a section or of a plate in compression, or the aggregate thickness of plates if connected together in accordance with Section 9, as appropriate

\( t \) = the thickness of the web of a section

\( t_{pe} \) = the thickness of the thinner of two components joined

\( W \) = the total load on a beam between supports

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Appendix 4

Extract from ‘Steel Designers Handbook’
by Gorenc & Tinyou
Maximum bending stress at point E:
\[
\sigma_E = -\frac{12 \times 10^6 \times 0.9962 \times 76.0}{8.32 \times 10^6} + 1.14 \times 10^6
\]
\[
= 106.6 + 49.4 = 156 \text{ MPa compression}
\]

Maximum bending stress at point F:
\[
\sigma_F = -\frac{12 \times 10^6 \times 0.9962 \times (-76.0)}{8.32 \times 10^6} + 1.14 \times 10^6
\]
\[
= -106.6 - 20.2 = -127 \text{ MPa tension}
\]

7.10 BENDING AND TORSION

7.10.1 General

When the plane of loading does not pass through the shear centre the beam bends and twists. The stress due to bending can be calculated separately from the stress due to twisting and then combined to obtain the total stress. To demonstrate the separation consider the section shown in Figure 7.37.

The section in bending only can have its stress calculated as described previously. The stress in the section due to the torque only can be determined by the methods of continuum mechanics or in very simple sections from tables of torsional properties. In some cases approximations for the bending stress due to the torque may be made simply. See Refs. 7.1, 7.3 and 7.7.

7.10.2 Twin-beam action

A method for approximating the additional bending stress in a beam due to torsion is presented here.

![Figure 7.37](image)

*Figure 7.37*  
Bending combined with torsion

The beam shown in Figure 7.38 has parallel flanges set apart some distance. Each flange has its ends connected to the support to prevent lateral movement, and hence providing torsional end restraint against twisting of the ends of the beam. A load \( P \), with an eccentricity, \( e \), to the shear centre causes a torque, \( P_e \), to be applied at some intermediate point along the beam, twisting it about the Z-axis. Resistance to twisting is afforded mainly by the bending rigidity of the two flanges with a smaller contribution by the torsional rigidity of the web. Each flange lies in a separate horizontal plane of bending which takes place about the vertical Y-axis. The horizontal loads, \( F_{yh} \), on each flange are equal and opposite to each other forming a couple, \( F_{yh} \), equal to the applied torque, \( P_e \). The web provides continuous lateral
Figure 7.38
Torque causing torsion in a beam, approximated by a couple acting on the flanges.

restrain to the flanges considered as beams bending about the Y-axis. If the flanges are wide, local buckling of the outer edges in compression is possible.

The requirements for satisfactory twin-beam action are:
(a) The ends of the beam are fully restrained against twisting. Some examples of how this may be achieved are mentioned in 7.8.6 and shown in Figure 7.25. The torque is applied at some intermediate point between the ends of the beam.
(b) Two parallel plate-like members spread some distance apart. One flange alone in a section is insufficient to attain twin-beam action.
1- and Z-sections and channels are the most common sections capable of twin-beam action. The resistance to twisting of beams with non-tubular sections by twin-beam action is only significant for doubly flanged members when compared with the resistance due to the thickness of material or the tubular nature of the section.

More accurate methods of analysis to determine the normal stress due to torsion are given in Refs 3, 9 and 11.
Example 7.12

Check the compressive fibre stress in a simply supported beam of span 8 m carrying loads eccentric to the shear centre as shown. The loads are applied at 2 m from one end. Each end of the beam is connected by angle cleats on both the top and bottom flanges to a column.

Method:

The maximum bending stress occurs at the point of application of loads.

\[ Beam~BDL = 82 \times 9.806 \times 10^{-3} \]
\[ = 0.8\, \text{kN/m} \]
\[ M_1 = 3.2 \times 2 - \frac{0.8 \times 2^2}{2} = 50 \times 2 \times 6 \]
\[ = 79.8\, \text{kNm} \]
\[ M_2 = \frac{2 \times 2 \times 6}{8} \]
\[ = 4.5\, \text{kNm} \]
\[ M_3 = 50 \times 0.015 + 3 \times 0.300 \]
\[ = 1.65\, \text{kNm} \]

Fibre stress due to plane bending

\[ f_{bd} = \frac{M_1}{Z} = \frac{79.8 \times 10^6}{2980 \times 10^6} = 26.9 \, \text{MPa} \]
\[ f_{bd} = \frac{M_2}{Z} = \frac{4.5 \times 10^6}{192 \times 10^6} = 23.4 \, \text{MPa} \]

Fibre stress due to torsion

This will be approximated by the method described in 7.10.2

The horizontal force acting one flange

\[ F_H = \frac{M_3}{h} \text{ where } h = 528 - 13.2 = 514.8\, \text{mm} \]
\[ F_H = \frac{1.65 \times 10^3}{514.8} = 3.2\, \text{kN} \]

Bending moment acting on one flange

\[ M_{1,2} = \frac{F_H \times ab}{L} \]
\[ = \frac{3.2 \times 2 \times 6}{8} = 4.8\, \text{kNm} \]
Elastic section modulus of one flange

\[ Z_{ef} = \frac{1}{8} Tyf \]
\[ = \frac{1}{8} \times 13.2 \times 209^2 = 0.096 \times 10^6 \text{mm}^3 \]
\[ F_{cet} = \frac{M_{et}}{Z_{et}} \]
\[ = \frac{4.8 \times 10^6}{0.096 \times 10^6} = 50.0 \text{ MPa. See note at end of example.} \]

Total \( f_{cet} = 23.4 + 50.0 = 73.4 \text{ MPa} \)

\[ \ell = k_1 A_1 l_1 \]
\[ = 1.20 \times 0.85 \times 1.00 \times 8 = 8.16 \text{ m} \]

\[ l = \frac{8160}{8.16} = 1000 \]

\[ r_e = 43.8 \]

\[ D = 40.0 \]

\[ F_{cet} = 66 \text{ MPa} \]

\[ P_{cet} = 0.66 \bar{F}_Y = 0.66 \times 250 \]

\[ = 165 \text{ MPa} \]

\[ f_{cet} = \frac{38.3 + 73.4}{66 + 165} = 1.02 \]

Conclusion

503 UB 82 is satisfactory for maximum compressive fibre stress.

Note: Compare with 25.6 MPa using data in Ref. 7.11.

### 7.10.3 Torsion

For a tube in pure torsion, the angle of twist per unit length of tube
\[ \theta = \frac{\varphi}{l} \]
is constant. In the above expression, \( \varphi \) is the total angle of twist due to a twisting moment \( M_t \) and \( l \) is the length of tube undergoing twisting.

The tangential shear stress in a thin-walled tube can be assumed to be constant through the wall thickness and can be computed from:

\[ f_t = \frac{M_t}{2 \pi r^2 t} = \frac{M_t}{2 A_t t} \]

where \( t \) is the thickness of the tube, \( r \) is the mean radius and \( A_t \) is the area enclosed by the mean circumference. The tangential shear stress \( f_t \) must not exceed the value of the maximum permissible shear stress.

The concept of the enclosed area \( A_t \) can also be extended to closed box tubes where:

\[ A_t = (D - \varphi) (B - t) \]

In this expression, \( D \) is the overall depth of the box and \( B \) is its width. The above expressions are valid only for closed sections such as tubes and box sections because
their sections after twisting remain in their plane within the practical limits of accuracy, and the torsional resistance contributed by the parts of the cross-section is proportional to their distance from the centre of twist.

When an open section such as an I-section is eg. flanges for example distort and tilt out of their plane and the above assumption is no longer valid. Unless such tilting (warping) is restrained, the torsional resistance contributed by each part of the cross-section is independent of its position from the centre of twist. This makes open sections substantially less rigid torsionally than box sections of the same overall dimensions and thickness.

The torsional rigidity of a member is \( GJ \) where \( G \) is the shear modulus and \( J \) is the torsion constant. A circular section, whether solid or hollow, is the only instance in which \( J \) takes the same value as the polar second moment of area. For other sections \( J \) is less than the polar second moment of area and may be only a very small fraction of it.

In a member under uniform torsion with none of the cross-sections restrained against warping, the torsional moment \( M_t \) is

\[
M_t = GJ
\]

For a box section of any shape and enclosing only one internal cell, \( J \) is given by

\[
J = 4 A_s^2 / \pi t^3
\]

where \( s/t \) is the length to thickness ratio of the component walls along the periphery of the section. In particular, a thin-walled round hollow section has:

\[
A_s = \pi d^2 / 4, \quad s/t = \pi d/4, \quad J = \pi d^4 / 32
\]

The torsion constant of a solid rectangular section in which the width \( b \) is some several times or more greater than the thickness \( t \) can be approximated by

\[
J = b^2 t^3 / 3
\]

where each \( b^2 t^3 / 3 \) is the torsion constant of an element. For example, \( J \) for an I-section becomes

\[
J = (2 BT^2 + ht^3) / 3 \quad \text{where} \quad h = D - T
\]

For an open section under uniform torsion, it can be shown that the maximum shear stress \( f_x \) is

\[
f_x = M_t / (J t)
\]

in which the maximum \( f_x \) occurs in the thinnest part of the cross-section.

For an I-section member under uniform torsion such that flange warping is unrestrained, the pattern of shear stress takes the form shown in Figure 7.39 (a).

---

Figure 7.39
Shear stress distribution:
(a) flanges free to warp
(b) flanges restrained against warping
When warping of the flanges is totally prevented, the shear stress through the thickness of the flange is practically constant and takes the distribution shown in Figure 7.29 (b). Under these conditions each flange is restrained against warping near rigidly in an ordinary rectangular beam carrying a horizontal transverse load. The most effective way of achieving complete restraint against warping of the flanges of an I-section is to box in the section by the addition of plates welded to the tips of the flanges.

Away from a location in which flange warping is restrained, the shear stress distribution is generally a combination of that shown in Figure 7.29 (a) and (b). The further the section of a beam is away from a location of warping restraint, the more is the distribution like that in Figure 7.29 (a). A rough approximation of the distance necessary from a position of flange warping restraint to a point along the beam in which the influence of restraint is negligible (and thus approaching the distribution in Figure 7.29 (a)) is given by

$$a = \sqrt{\frac{EI_c}{GJ}}$$

where

- $a$ = torsion bending constant
- $I_c$ = warping constant = $I_i h^2/4$ approximately for I-sections

The torsion bending constant $a$ is combined with the span $L$ into a dimensionless parameter $L/a$ for use in Tables 7.13 and 7.14.

Although box sections are more suitable, sometimes short stocky lengths of I-sections may be adequate in strength and stiffness for use in combined bending and torsion. If the span $L$ of a cantilever beam is less than $0.5a$, the pair of flanges act like twin beams loaded in opposite directions to take all the applied torsional moment $M_t$. For an I-section cantilever beam carrying a load at the free tip with an eccentricity $e$ to the centre of the beam, then from Figure 7.38, the horizontal force $F_h$ acting on each flange is given by

$$F_h = M_t e/h - Pe/h$$

Tables 7.13 and 7.14 include simple approximate expressions for the maximum flange moment $M_{fu}$ due to flange warping restraint and the maximum angle of twist $\psi$, for some common cases of torsional loading. Short and long beams are covered directly by the formulae quoted while beams of intermediate length require the use of coefficients which may be interpolated. See Ref. 7.16 for the interpolating polynomials.

Intermediate and long beams using open I-sections in combined bending and torsion are feasible only if the applied twisting moment $M_t$ is small. Large moments produce excessive twisting which become unacceptable.

More accurate solutions may be obtained analytically by using the differential equations:

$$M_t = GJ\phi'' - EI\phi''$$

and the appropriate boundary conditions to find the function $\phi$ involving hyperbolic or exponential functions. Standard solutions for the function $\phi$ for a variety of cases are published. Many aids are available to shorten the labour of computation.

Three idealized boundary conditions are possible:

(a) Free end in which the end of the beam is free to twist and also free to warp.

$$\phi = \phi' = \phi'' = 0$$

An example is the free tip of a cantilever beam.
Steel Designers Handbook

(b) Planned end in which the end of the beam is not free to twist but free to warp

\[ \phi = 0, \phi' = 0, \phi'' = 0 \]

Some examples are beam-to-column connections with reasonably deep web side plates, and flexible end plate connections.

c) Fixed end in which the end of the beam is not free to twist and not free to warp.

\[ \phi = 0, \phi' = 0, \phi'' = 0 \]

An example is a moment connection.

Charts are available for the direct reading of the values of the torsion functions \( \phi, \phi', \phi'' \) and \( \theta'' \) for given values of \( L/a \), and the position along the beam as a fraction of the span \( z/L \).

7.11 BENDING COMBINED WITH AN AXIAL FORCE

A beam which is also subjected to axial loading involves bending and axial stresses which can be combined using the intersection equations given in AS 1550, Section 8. Comments are included in Ref. 7.6 and another commentary is given in Ref. 7.8. Members carrying both bending moments and compressive axial loading simultaneously are known as beam-columns. For an example of the application of these equations, see Example 6.2 in Chapter 6.

7.11.1 Combined stresses

Any point in a structural member generally contains normal and shear stresses acting on different planes and in different directions. These stresses are the results of calculations carried out in some phase of the design of the member making use of the usual methods for determining the bending and shear stresses. When combined, these stresses produce maximum normal and shear stresses acting on other planes and in different directions. These maximum stresses should not exceed the permissible stresses. The combined stresses can be determined analytically or much more easily by Mohr's circle. In some cases such as beam-columns subjected to axial loading and bending together, the normal stresses due to the axial load and bending act in the same plane and direction, and they can be added algebraically to give the combined normal stress.

Example 7.13

A 200 mm thick wall 1.25 m high giving rise to a uniformly distributed load of 4.75 kN/m, is supported on a 250 UC 89 steel section as a simply supported spandrel beam of span \( L \) of 7 m. The wall is loaded to one side of the beam resulting in an eccentricity \( e = 100 \text{ mm} \) from the centre line of the beam.

Given:
The beam is attached by a deep web side plate at each end to the supporting column.
There are no intermediate lateral restraints.
Steel is grade 250.

Check the stresses in the beam and find the maximum twist in the beam.
Solution

1. Properties of section

Dimensions and section properties of a 250 considering, are:

\[ D = 260.4 \text{ mm} \quad B = 255.9 \text{ mm} \]
\[ T = 17.3 \text{ mm} \quad t = 10.3 \text{ mm} \]
\[ D \quad T = 15.9 \quad A = 11400 \text{ mm}^2 \]
\[ J = 1040 \times 10^6 \text{ mm}^4 \]
\[ I_x = 145 \times 10^6 \text{ mm}^4 \]
\[ Z_x = 1300 \times 10^6 \text{ mm}^3 \]
\[ I_y = 48.5 \times 10^6 \text{ mm}^4 \]
\[ Z_y = 379 \times 10^6 \text{ mm}^3 \]
\[ r_c = 65.3 \text{ mm} \]
\[ h = D - T = 260.4 - 17.3 = 243 \text{ mm} \]
\[ I_o = \frac{I_x}{4} \text{ approx. for 1-section} \]
\[ = 48.5 \times 10^6 \times 243^3 \times \frac{1}{6} \]
\[ = 715,969 \times 10^6 \text{ mm}^6 \]
\[ a = \sqrt{\frac{E T}{GoJ}} = \sqrt{\frac{200 \times 10^6 \times 715,969 \times 10^6}{80 \times 10^6 \times 1040 \times 10^2}} \]
\[ = 1312 \text{ mm} \]

2. Load

Self weight of beam = 0.87 kN/m
Wall load = 8.75 kN/m
Combined uniform load \( w = 9.62 \text{ kN/m} \)

3. Bending moments

The critical section for both the vertical bending and the horizontal bending due to torsion is located at mid-span.

\[ M_x = \frac{wL^2}{8} \]
\[ = \frac{9.62 \times 7^2}{8} = 58.9 \text{ kNm} \]

Each end of the beam is torsionally pinned because the web side plate prevents twisting so that \( \phi = 0 \), and the absence of connections between the beam flanges and column means that the end cross-section is free to warp so that \( \psi = 0 \) and \( \psi' = 0 \).

In a beam with a doubly symmetric cross-section, the centroidal axis is also the shear centre axis about which a beam in torsion twists.

\[ \frac{L}{a} = \frac{7000}{1312} = 5.3 \]

Therefore \( 2.6 < \frac{L}{a} < 8.0 \)
Appendix 5

Extract from
‘Review of Planning and Building Controls’

The Treasury - 1983
REVIEW OF PLANNING AND BUILDING CONTROLS

This is a discussion document prepared by the Review Team examining Planning and Building Controls. Its purpose is to set out the background to present controls, the issues seen as current, the various approaches to reform of present controls which might be taken, and how the future systems might be developed. All concerned are invited to make submissions on the various aspects, whether included in the document or not.

Wellington, New Zealand, 31 May 1983

Submissions will be welcomed, preferably by 30 September 1983. The Reviewers are available to discuss the document with interested parties.

Address all submissions to:

The Reviewers,
Office of the Review of Building Controls,
C/- The Treasury,
Private Bag,
Wellington,
NEW ZEALAND.
7.6 We have enquired into the significance of "public" health and safety as contrasted with "private" health and safety. While we understand the purpose of building controls extends to ensuring good standards of health we question whether the controls are really aimed at this or aimed at ensuring a reasonable and "marketable" design. If a person wished to design an unusual residence, e.g. with bedrooms connected by one metre diameter tubes or a dining area with a 2 metre ceiling height, should he be prevented from doing so? Most prudent owners have an eye to a future market, and accept or provide a design for building accordingly. Should not the market forces be allowed to influence building design more than they can now?

7.7 If a building owner desired to alter the interior of his residence by removing bearing partitions to the danger of himself and his family in the next storm, should he be prevented from doing so by the by-laws which say he cannot without a permit? At present he is prevented from doing anything to a partition, whether bearing or not, unless he has a permit to do so. His castle is not his own.

7.8 Chapter 4 of NZS1900 is quite specific in many respects as to minimum requirements for residential buildings. We find that the market has had such a strong influence on building design that probably every residence built in recent years is above the minimum requirement in most respects.
Appendix 6

Article from NZ Herald on Slump in Apprentice Skills
1 October 2004
Apprentice skill slump puts NZ out of world contest

**JOBS:** Competition head says trainees are being pushed too quickly and are not learning their crafts properly

by Stuart Dye

Trade skills are at such a low ebb that craftman apprentices are no longer taking part in a leading international competition.

New Zealand has had a strong reputation over the 50-year history of WorldSkills, the Olympic Games of vocational training.

But in recent years performances have slumped and organisers have been unable to find trainees of a high enough standard to send to the competition.

Allan Lund, chief executive of Youth Skills, which administers trade competitions, said this indicated a worrying trend of forcing more apprentices through the system, but leaving them ill-equipped to enter the workforce.

People today complained that they could not get a plumber, but the concern was that tomorrow they would get one who was not up to the job.

New Zealand youngsters had consistently finished in the top third in the world until the mid-1980s.

"Unfortunately, those days are not seeing evidence of that ability, which suggests training standards are slipping and our training systems are not keeping pace with the rest of the world."

Carpentry, plumbing, bricklaying and electrical wiring were the main areas of concern in the building industry, but the trend was also seen in other skill categories.

Plumbing was withdrawn from national competition in 1990, no carpenters had met the international standard since 1997 and the bricklaying category did not run last year.

Until 1997, New Zealand representatives in the world finals for electrical wiring had finished at lower than 10th, but had written two silver medals and three diplomas of excellence. Since then, no competitor had been good enough to qualify.

Mr Lund said apprentices were being trained on equipment used in manufacturing or assembly-line processes.

But they were not learning the traditional skills of a craftman.

The Government said this week it was spending $20 million over four years to expand its "modern apprentice-ship" programme.
Appendix 7

John Scarry Comments on Proposed NZTA Composite Bridge Design Guide
14 January 2011

Heavy Engineering Research Association
17-19 Gladding Place
Manukau City

Attention – [Name]

Dear [Name],

Comments on Draft – 23/12/2010
Steel/Concrete Composite Bridge Design Guide
NZTA Research Report TAR 09/04

1. Although I am not a bridge engineer, a copy of this draft report, which is clearly in an early form, was sent to me. I had a brief look through, and certain things that caught my eye which I think need correction/expansion. Hence these comments.

2. An Important Note for the Reader
I suspect this is a standard NZTA section, but the last sentence needs changing. Why on earth, for a report that is solely related to the profession of structural engineering, should the reader seek “….appropriate legal or other expert advice.”

It should read “….appropriate engineering or other expert advice.…”

3. Miscellaneous Typos
- In Acknowledgements – Charles Clifton
- Section 2, page 7, second paragraph, 2nd line – being (?), not ‘bring’
- Section 2.6.3, third line – cost, not ‘coat’
- Section 6.4, second paragraph, 6th line – following, not ‘follow’
4. **2D Grillage Analyses**

(a) **Torsional Stiffness of Equivalent Beams**
In Section 6.3.1, page 50, the stated torsional stiffness of $K = bt^3/6$ is theoretically correct but wrong for all practical purposes. If one has the results of a grillage analysis, and the ‘beam strips’ have equilibrium torques in them, how can one design for the torsion? One can’t.

Therefore, the approach of Hillerborg, with his manual strip method, should be followed. That is, $K$ is set to a very small value (but not so small as to cause ill-conditioning), such that all torques are close to zero.

Then, for an orthogonal grillage, the moments represent a safe lower bound, all in equilibrium, and design of the reinforcing is straightforward.

(See (e) below).

(b) **Skew Grillages With Orthogonal Reinforcing Layouts**
Requirements as to how to combine non-orthogonal bending moments to determine reinforcing should be given.

(c) **Shear in Grillage Analyses**
Although one way action is unlikely to govern, I think it should be checked. The shear in the beam strips in one direction represents the rate of change of moment in that direction, the shear in the other direction represents the rate of change of moment in the other direction, and a vector sum (adjusted for width?) should be checked.

(d) **Finite Element Analyses**
The classic plate equation for equilibrium is:

$$\frac{\partial^2 m_x}{\partial x^2} + 2 \frac{\partial^2 m_{xy}}{\partial x \partial y} + \frac{\partial^2 m_y}{\partial y^2} = -q$$

At any point in a slab, there will be a principal major moment ($M$ say) and a perpendicular minor moment ($m$). These moments have no twisting ($m_{xy}$) ‘attached,’ and the moment vectors are at variable alignments all over the slab.

With an orthogonal reinforcing arrangement, at each point, the moments **in the direction** of the reinforcing are a combination of the orthogonal moments ($m_x, m_y$) and the twisting moment ($m_{xy}$). The Wood-Armer equations developed in the 1960’s, which take the form

$$m_{ux} = m_x + |m_{xy}|$$

must be used to determine the reinforcing demand **at each point**.

For a simple non-skew slab, $m_{xy}$ will be small except in the corners, as expected. Unfortunately, it has been my experience that for slabs analysed using finite elements, even professional *analysts* simply print out diagrams of $m_x$ and $m_y$, and ignore $m_{xy}$. This is completely wrong, especially in those areas where skew and complexity require finite elements to be used.
Some FE programs will calculate and provide the Wood-Armer moments. The problem with this, of course, like \( m_x \), \( m_y \) and \( m_{xy} \) themselves, is that these values represent a moment demand of so many kNm/m width \textit{at a point}.

What the engineer needs is a moment demand of so many kNm over a reasonably accurate width of slab.

Some programs allow the definition of design strips, such that the total moment demand in that direction over the width of the strip is provided.

Therefore, I believe any such design guide must explicitly cover the following at least:

- The simplest of fundamental theory, and the Wood-Armer equations or similar, to leave no doubt as to how the design moment demand must include the demand of the twisting moments \( m_{xy} \).
- Detailed guidance is needed on how to deal with the ‘peaks’ that often occur in FE analysis, and more importantly, how to convert the infinite number of Wood-Armer ‘point’ moments into a practical moment over a given width.

(e) \textbf{Hillerborg and the Classic Equation}

The theoretical basis for Hillerborg’s strip method is that in the classic plate equation shown above, \( m_{xy} \) can be safely set to zero, then the equation becomes

\[
\frac{\partial^2 m_x}{\partial x^2} + \frac{\partial^2 m_y}{\partial y^2} = -q
\]

5. \textbf{Precast Concrete Decking Formwork}

In practice, it is almost certain that nearly all of the steel composite bridges designed and constructed will incorporate precast concrete decking formwork. However, although precast units are mentioned extensively, most of the slab, composite and shear connector provisions in the guide are based upon the decks being solid in-situ slabs.

This is a major weakness, and I believe there must be extensive diagrams and explicit design information and details dealing with all the issues caused by precast deck units.

(The significant lack of comprehensive design and detailing guidelines, and much entrenched bad practice in the use of precast floor units with steel and concrete beams in buildings, leads me to say this.)

I take it that these precast units are prestressed 100mm thick and 1200mm wide, like Stahlton’s 100mm thick ‘Flat Slabs.’

Therefore, assuming say 100mm thick precast panels with 150mm topping as an example:

- Minimum seating details should be shown (on mortar beds?),
- Resulting transverse bending on the top flanges, and torsions, due to wet concrete, etc, should be dealt with in detail,
- Any effect that precast slabs have on the transfer of vehicle loads to the beam shear centre should be covered, e.g. edge beams,
For the composite girders shown in Fig. 2.1, the concrete that can act compositely is a 100mm high section between the precast units, and then a 150mm thick slab ‘flange’ on top. This must be explicitly covered; I know many engineers who would assume a 250mm thick ‘flange.’

Again, with reference to Fig. 2.1, the deck will be analysed and designed as if a solid two way slab, yet, even with full composite action of the precast units in the ‘girder to girder’ direction, parallel to the girders the ‘slab’ is essentially only 150mm thick, because of the gaps between the precast units.

If welded triangular ‘trusses’ are to be incorporated into the precast units to enhance the span capacity during construction, as is often done overseas, this should be explicitly covered, with design guidelines.

Composite action between the precast units and the topping, especially minimum tie requirements, should be covered.

With regard to punching shear, the ‘d’ in one direction will be significantly larger than the ‘d’ in the other direction, because of the precast units, the gaps, and the arrangement of prestressing and deformed reinforcement. How does this affect the punching shear requirements?

If punching shear stresses are high such that shear reinforcing will be required, how can such reinforcing be installed, because the precast slabs are ‘in the way’?

When a shear stud is near a free edge (either in an edge girder close to the slab edge, or if cracks form between the precast units and the in-situ concrete), anti-splitting reinforcing is required, approximately 30mm above the bottom of the slab, if the studs are to have any chance of developing their design strength. (Refer Elementary Behaviour of Composite Steel & Concrete Structural Members, by Oehlers and Bradford).

However, precast units make it almost impossible to install this anti-splitting reinforcing. I think there needs to be comprehensive guidance and drawings dealing will all of these premutations and combinations.

6. Punching Shear Reinforcement

I must state that I do not possess, and avoid where possible using, NZS 3101:2006. It should never have been released, and needs to be withdrawn as soon as possible. Therefore, I am not familiar with its exact punching shear reinforcement requirements.

However, one thing that needs to be made clear is that for punching reinforcement in thin slabs, the full concrete resistance cannot be mobilised if conventional stirrup type reinforcement is used. This is because the slip in the stirrups, especially at their bends, over the short lengths involved leads to cracks that significantly reduce the concrete resistance.

Therefore, only something like $\frac{1}{2} V_c$ should be used with the $V_s$ of the stirrups.

(For typical beams, the length of the stirrup is such that the ‘strains’ are smaller, and full $V_c$ can be used).

Headed studs (such as lengths of rebar friction welded to bearing plates at each end), however, have very little slip, and, subject to certain limits, I think full $V_c$ can be used.

This needs to be covered in detail, I believe. The numerous papers by Amin Ghali should be consulted to ensure correct guidance is given.
However, I repeat what I said above:

- With regard to punching shear, the ‘d’ in one direction will be significantly larger than the ‘d’ in the other direction, because of the precast units, the gaps, and the arrangement of prestressing and deformed reinforcement. How does this affect the punching shear requirements?
- If punching shear stresses are high such that shear reinforcing will be required, how can such reinforcing be installed, because the precast slabs are ‘in the way.’

7. Fatigue in Reinforcing

NZS 3101:1982 used to have some (very small) reference to fatigue in reinforcing, e.g. that very large diameters should be used to form hooks in rebar.

Especially if shear reinforcement is required to prevent punching failure under wheel loads, should fatigue in main and shear reinforcing be covered?

8. Deck Diaphragm Behaviour

(a) Whereas temporary top chord bracing for restraint is covered, as is permanent bottom chord bracing for buckling or curvature restraint, I did not notice any mention of deck diaphragm design to resist seismic and other lateral loads. This may be assumed as a ‘given’ in NZ, but the following example shows that it isn’t, along with several major building ‘shockers’ I have come across.

(b) A few years ago, I was working on contract at Opus International Consultants, working on buildings, but sitting near the bridge engineers. Opus had been engaged as something like ‘the Owner’s representative’ for a new two span, highly skewed bridge. Opus was charged with overseeing that the overall client requirements were achieved. Peter Worrall was the Opus Structural Manager handling the job, and can attest to what I say.

The highly skewed bridge consisted of large prestressed concrete box girders. At each abutment, there were numerous piles, with revetment in front. Two large, tall columns supported the bridge midway between the abutments. In plan, the deck was ‘wide and stiff,’ not flexible.

The bridge had been analysed and designed (and I use the words advisedly) by company A, and fully peer reviewed and approved (and again I use the words advisedly) by company B. The design engineer in company A was a New Zealander, trained in NZ, but he had worked on bridge design overseas for several years. A senior director of company A, a well known engineer, was also closely involved in the project.

One evening, Peter Worrall wandered over, and asked a bridge engineer and me if we could help him out. The bridge had been modelled by company A in 3D using beam members on SpaceGass, and included a deck grillage to model/distribute the design vehicle loading. A dynamic analysis had been carried out.

What could not understand in the documentation provided was why approximately 50% of the total transverse seismic load went into the two central columns, and only 25% into each of the abutments, with their numerous buried piles.
Finally, it clicked. The mode shapes of the deck in plan, instead of having sharp corners, had corners that ‘drooped,’ like tadpole tails. There was no rigid deck diaphragm (which should have been modelled using master/slave nodes). The grillage was just wobbling around, with the load going to the columns and piles on a tributary area basis.

I raised this with Company A’s design engineer and director, but they still “couldn’t get it.” Things went downhill from there.

(c) I believe the guide must have comprehensive guidance on bridge deck diaphragm stiffness, behaviour and strength.

(d) One problem, of course, is that for many bridge decks, the assumption of a rigid diaphragm and implementation through master/slave nodes is valid, and leads to an accurate distribution of forces to the supports. However, it provides little to no information on the internal actions in the diaphragm.

‘Rigorous’ strut and tie models can do this, but:

- Each load case requires a different model (which soon becomes onerous in complex diaphragms and walls in buildings),
- For complex diaphragms, often a preliminary FE analysis is required to determine main stress paths.

I have developed, although not rigorously proved, a ‘poor mans’ strut and tie method that allows all load cases to use only one model, allows the loads to follow any path they wish, and allows the direct calculation of tension reinforcing requirements. It assumes that there are numerous small bars not subject to massive localised shear transfer, so that bond stresses are not an issue, buckling is ignored, and it is assumed that compressive stresses are not near the crushing stress, so that compressive stresses at nodes do not have to be resolved into resultants.

To model the wall or floor diaphragm, it is broken up into a grid of orthogonal truss elements aligned with the reinforcing, each element representing 300mm to 500mm width of concrete. Each ‘square’ has two compression-only diagonal truss elements, arranged in an X pattern. Supports and loads are added. Openings are dealt with by simply omitting truss elements.

For each load case, compression stresses for each member can be checked, and tension reinforcing is simply determined from the tension force in each orthogonal member. No diagonal tension forces have to be resolved.

Please advise of any errors in this approach.

9. **Welded Girders and Weld Details**

It may be obvious, but I think mention should be made of using welded girders with smaller top flanges for simply supported spans and the like.

Given that this is a bridge design guide, I think there should be a discussion of weld types for different locations (butt welds versus fillet welds), the effect of weld type on fatigue performance, and ways that welds can have their fatigue performance improved through shot blasting, ‘peening’ (?) and the like.
10. **Eccentric Steel Connections**
Given the failure and/or near collapse of several buildings due to incorrectly
designed eccentric cleat connections, I think this should be explicitly covered.

In addition, various other eccentric connections should be discussed, and their
fatigue performance discussed as well. For example, the connections shown in Fig.
2.3 and Fig. 2.5. All too often, these eccentricities are ignored or poorly handled in NZ.

Similarly, reference to gusset plate design, and especially buckling, should be made.
This has been a cause for bridge failure overseas, under gravity loads alone.

I know that some of this is covered in the bridge and material codes, but so is “$M_b = \alpha_m\alpha_sM_s$,” yet that is included in the guide.

11. **Anchor Bolts and Related Baseplates**
(a) ‘Supports’ in the guide seem to be shown simply as diagrammatic rubber bearings.
Anchor bolts, and even shear keys, surely are an important component to be covered.

Perhaps the SESOC Design Guide – Anchor Bolts for Steel Structures by John Scarry could be referenced to cover this.

(b) Fig. 2.7 appears to show a four anchor bolt arrangement at the top of a column. This
appears to be a ‘fixed’ connection. In the main, however, most anchor bolt
arrangements would, I believe, be ‘pinned.’

Especially given the possibility of fatigue and low cycle fatigue failure, I believe
baseplates and anchor bolt arrangements should be covered in detail, including such
bridge classics as the ‘crane rocker’ (a machined curved plate under the girder) and
‘double baseplates,’ with long bolts projecting. This allows flexing and stretch of the
anchor bolt, reducing yield and fatigue issues.

12. **Section 7.6.9 Lateral Stability of the Bridge**
I disagree with a 1% lateral restrain force. It is too small. Must be 2.5%, or 2% at
least.

Years ago, the Japanese considered dropping their lateral restraint requirements,
especially for laced and battened members, down to 1%. The resultants tests were
failures, and they reverted to, I think, 2% restraint forces.

13. **Shear Stud Strength and Section 7.7**
It is very unclear as to what ‘Clause 6.6.2’ etc refer to? Are these AS 5100 clauses?

Yours faithfully,
John Scarry  CPEng
Appendix 8

Supermarket Collapse
Burnaby, British Columbia
1988
Supermarket Roof Collapse on Opening Day

Burnaby, British Columbia 1988
Appendix 9

Getting It Right The First Time
Engineers Australia – Queensland
October 2005
An examination of an industry problem as it applies to Queensland and recommendations for solutions and actions.

This report was prepared by an industry-wide Task Force.

October 2005
Contents

Let’s Get It Right 1

Section 1: Executive Summary 3-5

Section 2: The Problem 6-9

Section 3: A “Whole Of Industry” Approach 10-11

Section 4: Analysing the Ten Core Issues 12-33
  4.1 Project briefs 14
  4.2 Project delivery 16
  4.3 Professional ethics and standards 18
  4.4 Selection strategy and bidding philosophy 20
  4.5 Risk management 22
  4.6 Design management 24
  4.7 Design process 26
  4.8 Human resource capacity 28
  4.9 Technology 30
  4.10 Communication 32

Section 5: Recommendations for Industry Change 34-56
  5.1 Project briefs 36
  5.2 Bidding philosophy and a selection strategy for consulting services 39
  5.3 Project delivery 43
    5.3.1 Professional ethics and accountability 44
    5.3.2 Risk assessment 44
    5.3.3 Lack of integration and communication 47
    5.3.4 Design issues 48
    5.3.5 Rebuilding our human resources 49
    5.3.6 Making effective use of technology 51
    5.3.7 Communication across the industry 53
  5.4 Implementation strategy 55
    5.4.1 Vision for the construction industry 55
    5.4.2 Building stakeholder support and participation 55
    5.4.3 Managing the task 56
    5.4.4 Communicating and marketing the message 56
    5.4.5 Changing industry culture and behaviours 56
    5.4.6 Continuous improvement 57
    5.4.7 Everyone to play a part 57

Appendix: Industry Support 58-61
  Feedback 59

Bibliography 62-65

Copies of this report are obtainable from:
- Engineers Australia.
  Engineering House, 447 Upper Edward Street, Brisbane QLD 4000.
Let’s Get It Right

Dear Colleague

The quality of project design documentation in the building, engineering and construction industry has declined significantly over the past 15 to 20 years.

This presents a huge challenge to the industry – many participants are unaware of the full extent of:

- the negative impact of current practices
- the enormous cost penalties on the industry
- the angst generated by the existing adversarial culture

In 2004, the Queensland Division of Engineers Australia set up a Task Force to take up this challenge. The Task Force now includes 17 other major industry organisations – all with a will to improve the situation.

This final report is the first major output of the Task Force – the result of two years of intensive research and collaboration within the industry. It carefully analyses the problem and proposes a plan of action to drive change in a tangible and sustainable way.

“Getting It Right The First Time” incorporates feedback from the industry in Queensland on a draft report issued in May 2005. It can be adopted across the broad spectrum of industry participants. It will be a catalyst for improved standards, and the basis for a robust and sustainable industry in the future.

We encourage you to play a part in creating change for the better in our industry – personally and through your own industry body. Spread the message about this report Australia wide through your national counterparts.

The future of our industry is in your hands – be involved, have your say, be an advocate for what we believe is an essential change to our workplace, our industry and the nation’s built environment.

Melissa Griffith
Queensland President 2005
Engineers Australia

Peter Jorss
Queensland President 2004
Chairman of Task Force
Engineers Australia

On behalf of and with the support of the industry-wide Task Force, which includes representatives of the following industry bodies:

Peak construction industry bodies, especially:
Queensland Major Contractors Association | Civil Contractors Federation | Master Builders Association Queensland | Queensland Property Council of Australia | Australian Institute of Building

Representatives of the Queensland chapters of the following industry bodies:
Australian Steel Institute, and Queensland Institute of Steel Detailers | Air Conditioning and Mechanical Contractors Association | Institute of Public Works Engineers Australia | Association of Consulting Engineers Australia | Royal Australian Institute of Architects | Australian Institute of Quantity Surveyors | Building Designers Association | Australian Institute of Project Management | Queensland Law Society (Construction Committee)

Queensland and Local Government construction authorities, in particular:
Department of Main Roads | Project Services Division of the Department of Public Works | Brisbane City Council
Section 1: Executive Summary

Purpose of the report

The purpose of this report is to present to the engineering and construction industry in Queensland a plan of action for overcoming the problem of the declining quality of project design documentation.

The declining standard of documentation is a problem for the engineering and construction industry nationwide, even worldwide. This report has focused on the problem in Queensland. However, the results will be applicable nationwide with the support of the various stakeholders’ national organisations.

The problem is well documented and has been brought about by a range of complex and often related causes. It has led to significant financial losses to consultants, contractors, clients, the State and its taxpayers; an overall loss of quality in the end product; and an increase in disputes and variations. This, in turn, has fuelled an adversarial environment.

Left unaddressed, this situation will continue to erode the foundations of the industry, lessening opportunity, dampening innovation, damaging reputations and deterring new recruits.

Since 2004, an industry-wide Task Force has been working on the problem and its solutions – the recommended solutions are outlined in Section 5 of this report.

A draft of this report was circulated widely in May 2005, and feedback from all parties has been strongly supportive.

Taking account of feedback received on the draft, this final report can be used by the industry as a plan to reverse the problem and as a tool to achieve best practice project design documentation.

Aims of the Task Force

The aims of the Task Force were to:

~ Recruit whole of industry support
~ Understand the wider impacts of poor quality of documentation on the industry and on the State economy
~ Identify the full range of critical issues and their primary causes
~ Develop practical and sustainable recommendations that address the critical issues
~ Influence all parties along the supply chain to avoid the pitfalls and adopt best practice in project design documentation.

The problem

It is well known within the building and construction industry that the quality of project design documentation has declined significantly over the past 15 to 20 years.

The “No Disputes” report by a joint working party of the National Public Works Conference (NPWC) and the National Building and Construction Council (NBCC) in May 1990 identified many aspects of the problem and recommended solutions.

Professional scales of minimum recommended fees such as the Association of Consulting Engineers Australia’s (ACEA) fee scale were discontinued in the 1980’s, under pressure from the competition authorities, and since then consultants have had to bid for engineering design work. More often than not, the work has been awarded on price rather than value and capability.

Extensive research backed by the wide industry experience of members of the Task Force shows that:

~ The declining standard of project design documentation has a positive correlation with a 24% decrease in design fees over the past 12 to 15 years
~ 60 to 90% of all variations are due to poor project design documentation
~ Standards continue to decline (a common perception – backed up by CSIRO research)
~ The industry strongly supports finding a solution to the problem
~ Poor documentation has led to:
  ~ an inefficient, non-competitive industry
  ~ cost overruns, rework, extensions of time
  ~ high stress levels, loss of morale, reduced personal output
  ~ adversarial behaviour, diminished reputations

Anecdotal evidence indicates that this year (2005) Queensland is influencing the method of selection of consultants; there is also recognition of many and valid forms of contract – Main Roads, for example, is trialling “Early Contractor Involvement”; but there remains an overriding need for improved quality of documentation.
The cost of leaving things as they are:

- Poor documentation is contributing an additional 10-15% or more to project costs in Australia
- The annual cost to the Queensland construction budget is a financial loss of $2 billion every year! This equates to a loss Australia wide of at least $12 billion.

Ten root causes behind the problem

The Task Force identified a plethora of industry issues underlying the problem. An analysis of the issues resulted in the identification of 10 root causes:

1. Inadequate project briefs based on unrealistic expectations of time and cost
2. Lack of integration along the supply chain linking the parties, and between project phases
3. Devaluing of professional ethics and standards of business practice
4. Lowest bid selection strategy rather than value for money
5. Poor understanding and skilling in risk assessment and management processes
6. Absence of an experienced client-appointed, overall Design Manager / Coordinator
7. Poor understanding of optimised and properly documented designs
8. Inadequate availability of, and recruitment of, skilled and experienced people
9. Inadequate/ineffective use of technology in design and documentation (e.g. poor application of CAD techniques; technical specifications drawn from an organisation's database but not tailored to the particular project)
10. Lack of appreciation of the benefits of open communication

Recommendations

To address these causes and the conflicting and overlapping issues the industry faces, the Task Force has grouped its recommendations into four categories:

1. Project briefs – including:
   - Risk assessment and allocation
   - Communication practices

2. Bidding philosophy and a selection strategy for consulting services

3. Project delivery – including:
   - Professional ethics, standards and accountability in business practices
   - Risk management
   - A client-appointed overall design manager
   - Design process – what is required to optimise designs and provide quality documentation
   - Adequate human resources
   - Adequate and effective use of technology, e.g. CAD
   - Effective communication practices

4. Implementation strategy – including:
   - Vision for the construction industry
   - Building stakeholder support and participation
   - Managing the task
   - Communicating and marketing the message
   - Changing the industry culture and modifying behaviour
   - Continuous improvement
   - Everyone to play a part
Remedial activities that apply to most of the 10 root causes are:

1. Making sure that the whole of the industry and all of its players are fully aware of the current situation, the consequent waste of resources, and an increasing difficulty with the “do nothing approach”.

2. Promoting the need for a change in culture, across the industry.

3. Reinforcing existing codes of practice, guidelines for industry, and legislation and regulation (modifying where necessary and in some cases assembling new).

4. Strengthening the requirements of continuing professional development, tertiary training and in-service/on the job training.

5. Reinstating equitable remuneration for professional design services within a value for money equation.

6. Improving project delivery across the board.

Actions

1. An industry-wide effort required

This report sets out the issues, the root causes and the draft recommendations in a no-blame non-adversarial manner. For resolution of the problem, however, every sector of the industry needs to play a part:

<table>
<thead>
<tr>
<th>Industry Participants</th>
<th>Part to Play</th>
</tr>
</thead>
</table>
| **Project Owners**    | • provision of project briefs  
                       | • consultant capability and value ahead of lowest fees  
                       | • effective risk management |
| **Professional Consultants** | • competent deliverables ahead of the lowest fee  
                                | • professional ethics  
                                | • recruitment and on-the-job training |

| Constructors and Suppliers | • competent performance ahead of the low bid  
                            | • cooperation ahead of adversarial behaviour  
                            | • recruitment and on-the-job training |

| Financiers and Lawyers | • a fair contract between equals rather than a tight and outdated master servant contract  
                        | • cooperative approaches rather than adversarial |

2. A win-win outcome for all

The Task Force strongly recommends that all parts of the industry emphasise value for money within a culture of cooperation and excellence, rather than focus on the lowest competitive bid within an adversarial climate.

3. Rebuilding our industry – support this effort

The support the Task Force has received means there is widespread interest in and demand for change that will build this win-win outcome.

As we have outlined here, there are immediate steps individuals and organisations can take and there are further steps that will take planning and action.

We urge every industry participant to:

• start the process of change today;
• change your approach to project design documentation; and
• become involved in the efforts of your industry association to make the industry-wide and larger changes required to make this better, safer, value for money approach the norm in our industry.

Please take time to read this report and make a start on the measures that will build a better outcome for all.
Section 2: The Problem

Falling standards of project design documentation

A common perception in the building and construction industry is that the quality of project design documentation has declined significantly over the past 15 to 20 years. The "No Disputes" report by a joint working party of the National Public Works Conference (NPWC) and the National Building and Construction Council (NBCC) in May 1990, for example, identified a number of aspects of the problem and recommended solutions.

Professional fee scales such as the Australian Consulting Engineers Australia's (ACEA) scale of minimum recommended fees were discontinued in the 1980's under pressure from the competition authorities. Consultants have had to bid for engineering and architectural design work and, more often than not, the work has been awarded on price rather than value and capability.

L Wilson (Construct in Steel, Vol 34, No 4 December 2000) discusses views recently expressed by Janet Holmes à Court (Chairman of John Holland Construction):

"I see the poor quality or late delivery of design documentation on every single one of our projects, with architects encouraged (by clients) to allow builders to start building before all the drawings are quite ready. It starts from day one and everyone gets into strife. On every project where we have difficulties, I can guarantee that design documentation is the main source of the problem."

On the other hand tendering on sketch plans of partially completed documentation for major projects is routinely adopted by many client organisations to transfer design risk to the managing contractor. Nevertheless, the problem is particularly evident in the building and infrastructure sectors – more so than in the resources sector. Perhaps this is because the owners of resources projects are more concerned with the "whole of life" performance of their projects, for example, ease of maintenance, and reliability throughout the life of the project, the project output, the commonality of spares, and operational safety.

In the mining sector:

bullet Client decisions are made by professional engineers
bullet There is a higher level of repeat business for consultants
bullet Design fees are less critical than delivery on time

The evidence

Extensive research into the quality of project design documentation backed by the broad industry experience of the Task Force members shows that:

bullet A CSIRO industry survey found that 68% of designers and 88% of contractors felt that documentation quality had declined over the past 12 to 15 years and that real design fee incomes had declined approximately 24%. (*refer Note 1)

bullet Design efficiency has a nonlinear inverse relationship with project design fees. (*refer Note 1)

bullet Project costs due to design inefficiency increase sharply when design fees are reduced below the cost of doing work properly. (*refer Note 1)

bullet The concept of reducing total project costs by increasing expenditure on the design process has been well-documented through principles of value engineering and value management. (*refer Note 1)

bullet 60 to 90% of all variations are due to poor project design documentation. (*refer Note 2)

bullet One price variation results from every three Requests for Information (RFI's). (*refer Note 2)

bullet Poor documentation contributes, on average but conservatively, an additional 10-15%* to project costs. (*refer Notes 3 and 4)

Total Queensland Construction Budget 2005

<table>
<thead>
<tr>
<th>Total Queensland Construction Budget 2005</th>
<th>20.7 bn</th>
<th>8.7 bn</th>
<th>$5.8 bn</th>
</tr>
</thead>
<tbody>
<tr>
<td>Estimated waste due to standard of project documentation</td>
<td>$3.0 bn</td>
<td>$2.0 bn</td>
<td></td>
</tr>
<tr>
<td>Multiplier effect on QLD economy (2.9x)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Note: Data and statistics may vary and should be verified through current research and studies.
• Substandard project documentation therefore equates to an annual financial loss exceeding $2 billion in the Queensland construction budget – every year! (*refer to Notes 3 and 4)

• Standards continue to decline (This is a common perception – backed up by CSIRO research.
  Refer note 5 for example):
  - poor documentation leads to serious underperformance within the industry, and:
  - a significant waste of resources – money, materials, time and labour
  - an inefficient, non-competitive industry
  - cost overruns, rework and extensions of time
  - high stress levels, loss of morale and reduced personal output
  - adversarial behaviour and a lowering of professional reputations
  - a potential decline in safety standards (*refer to Note 6)
  - a decline in the viability and sustainability of the industry

Proper project design documentation should be:
• fit for purpose
• unambiguous and coherent
• timely, accurate and complete
• easily communicated and constructed, with the best possible economy and safety
• aligned with the owner’s requirements as set out in a project brief

Notes

[*Note 1: PA Tilley, SL McFallan and SN Tucker, “Design and documentation quality and its impact on the contract process”, CSIRO, IEAust Dec 2000, in a survey of 5500 design consultancy firms and contracting firms, with an overall response rate of 5.4%]

[*Note 2: Private communication from Eric Levett, Project Services]

[*Note 3: For example, an analysis of a sample of 50 projects at a leading Queensland Government building authority showed that 88% of variations were due to design and/or documentation problems rather than client requested changes.]

<table>
<thead>
<tr>
<th>Source of Costs</th>
<th>Percentage of Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cost of variations</td>
<td>= 88%** of total variation cost of 14% of total contract value (projects totalling $200m) = 12% of total contract value, plus</td>
</tr>
<tr>
<td>Cost of administration</td>
<td>= 0.9% of total contract of variations value, plus</td>
</tr>
<tr>
<td>Cost of extensions of time</td>
<td>= 2.1% of total contract time (est.) value plus</td>
</tr>
<tr>
<td>Cost of administration of EoT’s</td>
<td>= 0.2% of total contract value of EoT’s</td>
</tr>
<tr>
<td>**Total cost (upper estimate)</td>
<td>= 15.2% of total contract value</td>
</tr>
</tbody>
</table>

** at a conservative 60%, in lieu of 88%;

| Total cost (lower estimate) | = 12.0% of total contract value |

[*Note 4: Australia’s Cooperative Research Centre For Construction Innovation (established in 2001 and headquartered at the Queensland University of Technology) quotes:
  “In 1996-7, the construction cluster comprising supply network, property sector and project-based firms contributed 14.4% to Australia’s GDP through 230,000 firms employing 730,000 people.
  The Australian Bureau of Statistics estimates that from an initial $1 million extra output and construction, a possible $2.9 million in output would be generated in the economy as a whole.
  Poor quality design and documentation is estimated to cost 7% of total construction costs with rework in construction likely to be greater than 10% of project cost. A 10% improvement in efficiency in the construction industry could boost GDP by 2.5% over five years.”


[Note 6: John Carpenter, Secretary of SCOSS, claims:
  “There is considerable evidence of declining standards in the communication of structural information to the contractors.
  Not only does this lead to cost overruns and dispute, it also raises the spectre of safety being compromised”. (The Structural Engineer; 3 August 2004.)]
Critical issues

The first step taken in tackling the problem was to list the critical issues currently faced by the industry. The list of issues is based on research and the extensive industry experience of the members of the Task Force. These have been categorised under generic “no blame” headings:

1. Issues relating to the current industry environment:

   **Culture**
   - Adversarial attitudes embedded in the industry
   - Lack of interaction between client, designer, contractor and supplier
   - Lack of understanding of the impact of decisions by upper management, e.g. antipathy towards relationship contracting for design and/or construction services

   **People, skills and training**
   - Erosion of expertise including inadequate training and mentoring
   - Lack of funds for on-the-job training and retention of staff between projects

   **Competition**
   - Reluctance to allocate experienced staff to projects because of their high cost in an environment of low-bid fees which are insufficient for the services required
   - Competition and purchasing policies inappropriate to complex design and/or construction projects, e.g. effects of state purchasing policies and/or their implementation
   - Competition legislation focusing on price rather than on Qualification Based Selection
   - Focus by client on low price driving price behaviour by consultants and contractors

   **Risk allocation**
   - Unsupportable risk allocation philosophy
   - Adversarial approach to variation of costs or time

2. Issues relating to project initiation

   - Incomplete project brief and changes to the brief
   - Insufficient project business case (net present values, whole-of-life analysis, assessment of benefits)
   - Unresearched / inaccurate time and cost expectations
   - Fewer greenfield projects and a lack of understanding of the extra risk on brownfield, or retrofit projects leading to tighter tolerances, more interfaces and increased costs without corresponding recompense in fees

   - Lack of client understanding of impact of their decisions

3. Issues relating to project development

   - Inappropriate dependence on price competition at the design stage
   - Unrealistic expectations about time and cost restraints on achievement of overall project outcomes
   - Lack of leadership in monitoring project outcomes
   - Lack of overall project coordination, e.g. between design disciplines
   - Reluctance to formally commit to a design at the appropriate stages of the project
   - Unrealistic project development design times
   - Insufficient and/or late review of design and checking of project design documentation
   - Insufficient review with relevant parties
   - Incomplete and/or erroneous design and project design documentation, in particular, lack of accuracy, clarity and timeliness – leaving design issues to be sorted out in the construction process

   - Inadequate application of design experience and knowledge to CAD design techniques

   - Lack of appreciation of tighter tolerances and extra interfaces on brownfield sites
   - Inadequate and/or competitive fees limiting proper allocation of experienced staff time and limiting capacity to hold staff between projects

   **Project planning**
   - Lack of appreciation of erection or installation sequence and erection loads
4. Issues relating to project delivery

Methodology

- Optimum design solution not adequately researched
- Inappropriate downward re-negotiation of tendered prices pre-award – both head contract and sub-contract
- Unrealistic tender periods and multiple extensions of closing date
- Unrealistic and/or unclear insurance requirements

Adversarial as against non-adversarial format

- Defensive approach to variations and claims for additional costs or time
- Adversarial attitudes embedded in the building and construction industry in general and in company cultures specifically

Detailed issues

- Excessive “Notices to Tenderers” and extensions of tender closing date, and the absence of a documentation freeze period prior to tenders closing to allow an adequate review period
- Failure to carry out and/or take responsibility for subsurface investigation
- Failure to provide adequate set-out data
- Outdated material and/or plant specifications
- Excessive number of Requests for Information and inefficient management of the RFI process

5. Issues relating to project life

- Inadequate consideration of “whole-of-life” issues
- Fitness for purpose
- Ease of operation and maintenance
- “Gold-plating” of industry documentation and/or inappropriate standards for the required life of project
Section 3: A “Whole Of Industry” Approach

In December 2002, a paper entitled “Project Documentation Quality and its impact on the Building and Construction Industry” was presented to Engineers Australia in Brisbane by Gallo, Lucas, McLennan, Parminter and Tilley.

Early in 2004 the Queensland Division of Engineers Australia set up a Task Force to investigate the problem of falling standards of contract documentation as discussed in Section 2.

This report has been prepared by this task force, sponsored by the Queensland Division of Engineers Australia and fully supported by representatives of the Queensland chapters of the following industry bodies:

Peak construction industry bodies, especially:
- Queensland Major Contractors Association
- Civil Contractors Federation
- Master Builders Association Queensland
- Property Council of Australia
- Australian Institute of Building
- Australian Steel Institute, and Queensland Institute of Steel Detailers
- Air Conditioning and Mechanical Contractors Association
- Institute of Public Works Engineers Australia

Peak industry consultant bodies, including:
- Association of Consulting Engineers Australia
- Royal Australian Institute of Architects
- Australian Institute of Quantity Surveyors
- Building Designers Association
- Australian Institute of Project Management
- Queensland Law Society (Construction Committee)

Queensland and Local Government construction authorities, in particular, both as clients and constructors:
- Department of Main Roads
- Project Services Division of the Department of Public Works
- Brisbane City Council

Letters of support for the Task Force’s effort have been received from the heads of most of the above bodies. This “whole of industry approach” adds weight to the recommendations in this report.

All members of the Task Force are volunteers drawn from all of the industry bodies listed above who have devoted a significant amount of energy and enthusiasm to this project. Collectively, there are some 900 years of industry knowledge, experience and capability among the Task Force members.

The Task Force’s aims were to:

- Understand the wider impacts of poor quality of project documentation on the building and construction industry and on the Queensland economy, and on the well-being and profitability of all industry players along the supply chain.
- Articulate the problem in a way that allows identification of the full range of critical issues and their primary causes.
- Develop recommendations that address the critical issues, and that can be given practical effect in a tangible and sustainable way.
- Influence the industry at all points along the supply chain to avoid the pitfalls of and to consistently adopt best practice in project documentation quality.

Scope of the Task Force’s investigation and report

The declining standard of project documentation is a problem for the building and construction industry nationwide, even worldwide. This report focuses on the problem in Queensland – a more manageable exercise eliminating the need for a significant budget, extensive travel and a secretariat.

We anticipate the results will be applicable nationwide with the support of various stakeholders’ parent organisations.

Previous research

The primary aim has been to identify and implement solutions that overcome the industry problem of the poor quality of project design documentation. No original research was undertaken – instead the approach has been to review the numerous existing research papers and the outputs of past and present industry reform initiatives such as:

- “No Disputes” put together jointly by the National Public Works Conference (NPWC) and the National Building and Construction Council (NBCC) in May 1990.
- The Construction Industry Development Association (CIDA) funded by the Federal Government in the early 1990’s.
- The CSIRO’s federally sponsored Cooperative Research Centre (CRC)’s “Construction Innovation” program in the early 2000’s.
• Work by the Australian Procurement and Construction Council, in conjunction with the Australian Construction Industry Forum (APCC+ACIF) in 2001-2002.

The Task Force members formally acknowledge the valuable work done previously by the industry. It would not have been possible to complete this task without standing on the shoulders of those who have researched and written about various aspects of the problem in the past.

The Task Force research focused on ensuring that the project outcomes were based on the best available information. The task was to bring all the known information into one place so it could be centrally reviewed and categorised for relevance and usefulness. Tools were then developed to make the information easily available to respective subgroups working on specific elements of the task.

Industry researchers were consulted in the identification of relevant research material. A Register of Research Material was developed, including more than 160 reviewed papers and over 60 books.

In addition to the research, each of the individual task force members has relevant experience in the compilation and/or use of project documentation and has made this experience available to the project – some 900 years of accumulated industry experience in total.

Methodology – an industry-wide line of attack

This report summarises the outputs from a logical series of activities:

i) definition of the problem

ii) listing of the issues

iii) a review of existing research

iv) derivation of a list of root causes, each selected as a practicable intervention level from a hierarchy of related issues by:

- analysing the causes and sorting them into “themes” or families of related causes
- arranging each family of causes into a hierarchy, i.e. from broad, general causes down to narrow, specific cause
- nominating the cause (or “intervention level”) within each hierarchy that has the maximum sustainable impact on the problem

v) matching of solutions developed by the group together with solutions extracted from the literature, to each root cause

vi) planning for the sustainable implementation of the report’s draft recommendations

vii) compilation of the draft report

viii) review of the draft report by interested parties across the industry and incorporation of feedback by the task force into a final report

ix) production and publication of a final report incorporating generally accepted recommendations

x) promotion and monitoring of implementation of the report findings in Queensland with, where necessary, legislative support

xi) adoption nationwide

An interactive construction forum in Brisbane in May 2004 gave universal support to the definition of the problem and the issues set out in this report.

A second interactive forum in June 2005 gave positive feedback to the report’s recommendations - as did individual feedback forms from industry participants. A total of 300 participants experienced in all sectors of the industry participated in the feedback process.

Good work in the past by various interested parties largely failed to make a difference to the industry. Much of this good work lacked whole of industry involvement which lacked industry ownership and support. Many efforts were government initiatives whose funding and support eventually withered.

To ensure this report does make a difference, its emphasis is on the support and involvement of the whole of the building and construction industry. This effort will continue to be on voluntary basis under an efficient budget funded by industry (hence not subject to the vagaries of government funding). Nevertheless, a government subsidy would be welcome to see this project through to a successful outcome.
Section 4: Analysing the Ten Core Issues

To find solutions the task force considered it essential to work from the causes that either in isolation or in combination, produce this multi-dimensional problem.

The starting point was to put all causes into groups with a common theme – a process that produced 10 distinct families. Within each family of causes some are more difficult to address – some are less difficult. Each of the groups was sorted into a hierarchy under which the top cause is the most difficult to deal with – the bottom cause is the least difficult to deal with.

Root causes and intervention levels

Analysis of the causes identified a number of root causes that promote and sustain the growth of the problem. These are shown on the charts in this section as the intervention level, this being the highest level at which the task force felt a practical response could be initiated.

Whilst each intervention level has become a focus in reversing the declining standard of documentation, the recommendations also address the issues above and below the intervention level.
The root causes identified by the Task Force are:

1. Inadequate project briefs based on unrealistic expectations of time and cost.
2. Lack of integration along supply chain linking the parties, and between project phases.
3. Devaluing of professional ethics and standards of business practice.
4. Lowest bid selection strategy rather than value for money.
5. Poor understanding and skilling in risk assessment and management processes.
6. Absence of an experienced client-appointed, overall design manager.
7. Poor understanding of optimised and properly documented designs.
8. Inadequate availability of, and recruitment of, skilled and experienced people.
9. Inadequate/ineffective use of technology in design and documentation (e.g. poor application of CAD techniques; technical specifications drawn from an organisation’s data base but not tailored to the particular project).
10. Lack of appreciation of the benefits of open communication.
Typically, the client/financier will identify a business opportunity and prepare a business case to prove the viability of the project before briefing the consultant to prepare designs and contract documentation. Whilst in the best cases, a project manager is appointed by the client to manage the delivery, in a high proportion of cases, the client/financier is overwhelmingly focused on the commercial or political outcomes and pays little regard to the downstream project delivery aspects, such as design detail, documentation and construction. In maximising narrow project benefits, unworkable demands and unattainable goals are imposed on both designers and constructors.

The practice of using “back briefs” or “return briefs” prepared by the consultant after discussion with the client is common in the industry - though less effective than commissioning a consultant for joint preparation of the brief.

Suzanne Wilkinson in her paper “Analysis of the Problems Faced by Project Management Companies Managing Construction Projects” outlined it in these words: “When dealing with clients, companies appear to be finding it difficult to obtain a client brief, understand the client brief and are finding that this is a poor brief definition. The development of a workable brief is seen as a problem, as the client must have a clear brief if the project is to be successful. The trouble is that many clients want a land-based flying submarine.”

Poorly prepared project briefs based on inadequate scoping and investigation are used to pass the responsibilities down to consultants who seem to be willing to take on the work on a low-bid basis.

The time and cost parameters set by the client are often based on wishful thinking rather than a thorough understanding of time and cost parameters required for effective delivery.

### 4.1 Project briefs

<table>
<thead>
<tr>
<th>Unrealistic project briefs</th>
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<tbody>
<tr>
<td>• Project initiated by political or commercial endeavour.</td>
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<tr>
<td>• Compressed timeframes because a high proportion of project time is used up in front end in establishing project viability.</td>
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<tr>
<td>• Unrealistic client expectations, particularly of time and cost due to poor client appreciation of cost drivers and project risks.</td>
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<table>
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<tr>
<th>Intervention Level</th>
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<tr>
<td>• Poor project briefs based on unrealistic expectations.</td>
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<tr>
<td>• Unrealistic expectations cause an imbalance in the (time/cost/quality/scope) equation.</td>
</tr>
<tr>
<td>• Poor communication/relationship with the client.</td>
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<tr>
<td>• Financial and time overruns.</td>
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<tr>
<td>• Disputes over scope.</td>
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</table>

**Poor quality of design and documentation**

- Typically, the client/financier will identify a business opportunity and prepare a business case to prove the viability of the project before briefing the consultant to prepare designs and contract documentation.
- Whilst in the best cases, a project manager is appointed by the client to manage the delivery, in a high proportion of cases, the client/financier is overwhelmingly focused on the commercial or political outcomes and pays little regard to the downstream project delivery aspects, such as design detail, documentation and construction. In maximising narrow project benefits, unworkable demands and unattainable goals are imposed on both designers and constructors.
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- Poorly prepared project briefs based on inadequate scoping and investigation are used to pass the responsibilities down to consultants who seem to be willing to take on the work on a low-bid basis.
- The time and cost parameters set by the client are often based on wishful thinking rather than a thorough understanding of time and cost parameters required for effective delivery.
Consultants in this situation seem prepared to compromise ethical behaviour by “cutting the cloth to suit the purse” in a “get what you pay for” operation that leads to dissatisfaction, conflict and a poor standard of project design documentation quality.

A vision for project briefs

- Project briefs for planning and development of all significant projects will be comprehensive and accurate and will permit all parties to properly assess the work required and to confidently make documents that all parties can rely on during the commission.
- Industry accepted models for good project briefs will be in common use across the industry.

This approach is well supported by the Australian Construction Industry Forum in its “Guide to Integrated Project Procurement” 06/09/01: “Having established the needs of the project..., the Client, in association with the user, must develop a well-considered, complete brief for the Project.”

Indeed, the Australian Construction Industry Forum has placed Client Brief and Project Establishment as Protocol No. 1 in its guideline document, “Improving Project Documentation”.

Objectives for proper briefing

- The relationships between the parties that are professional and based on a win-win philosophy.
- Administration and management that are conducted in good faith.
- Generous descriptions of the project context and its background including clearly defined project objectives and any special drivers for the client.
- Comprehensive scope definition and functional requirements.
- Procedures to finalise the brief.
- Clear expectations about cost and time including a reliable cost plan that recognises the links between scope, time, cost and quality and incorporates appropriate contingency.
- Deliverables specified including full description of engineering and architectural services required.
- Comprehensive stakeholder analysis and processes to include architectural, engineering and construction participation in the development phase.
- Clients’ project management arrangements.
- Management of permits and approvals including legislation constraints.
- Clients’ expectation about particular disciplines required.
- Client involvement, decision processes, scope change management.
- Inputs; documentation; information, e.g. surveys, geotechnical, data sheets.
- Communication protocols (reports, meetings, relationships).
4.2 Project delivery

There is often a lack of effective integration of and feedback between delivery phases, which in turn impacts adversely on constructability, rework and innovation.

During design phase specialised elements are often inadequately drawn together which results in difficulty in interpretation, lack of clarity and confusion with specification and contract conditions.

The Rethinking Construction Task Force (Sir John Egan, 1998) states that “…this is indicative of a fundamental malaise in the industry – the separation of design from the rest of the project process.”

This climate, coupled with inappropriate risk/reward arrangements, reinforce the adversarial relationships which only compound the problems.

<table>
<thead>
<tr>
<th>Lack of integration along supply chain</th>
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<tr>
<td>• Modern projects are frequently complex (e.g. brownfield sites) and greater societal expectations (e.g. environmental responsibilities).</td>
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<tr>
<td>• Often projects have multiple objectives and stakeholders.</td>
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<tr>
<td>• Minimal client/owner leadership (i.e. limited input and involvement, poor communications).</td>
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<tr>
<td>• Public sector agencies tend to use design then construct delivery methods.</td>
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**Intervention Level**

| • Poor integration along the supply chain, i.e. between project phases. |
| • Delivery process frequently breaks the project into separate stages and creates management “silos”. |
| • Poor coordination between project stages – inefficiencies, loss of opportunity and lack of innovation. |
| • Procurement methodology is based on win-lose terms of engagement. |
| • Internal competition between those responsible for particular elements (i.e. passing blame). |
| • Adversarial relationships. |
| • Constructability problems – delays and increased costs. |
| • Design rework in construction phase is costly and wasteful. |

Poor quality of design and documentation
Current status of project delivery

Modern projects are generally being delivered in a complex environment with diverse and competing expectations by the stakeholders and the community at large. There is less vertical integration within the supply chain. Typically clients are focusing more on initial total project costs and minimising input costs such as design, rather than minimising total implementation costs, let alone optimising whole-of-life project costs.

This view is supported by Love, Irani and Edwards in their paper “A Seamless Supply Chain Management Model for Construction”. For example: “The separation of the design and production process in projects has been widely criticised during the last fifty years or so, e.g. Simon (1994), Banwell (1964), Latham (1994) and Egan (1998).” and also “…calls for improved collaboration, integration, communication and coordination between customers and suppliers throughout the project supply chain have been the leitmotif of the published reports.”

Objectives for proper project delivery

- Generate information once only in a way that can be shared across disciplines and down the supply chain more easily. Information that can be used in the operation and maintenance stage of the infrastructure lifecycle as well. For example, this may well involve the use of virtual projects generated through the use of 3D and 4D CAD.

- Promote a range of procurement systems (with variations) appropriate to the circumstances of the project that share risks appropriately and compensate the parties for the risks taken.

- Emphasise, as far as possible, alliances and partnerships that are based upon the best outcome for the project overall and the mutual benefit of all contributors.

  “…by attaining maximum business process efficiency and effectiveness through inter and intra organisational relations” – Love, Irani and Edwards.

- Involve other participants in the supply chain in the earlier stages, not just contractors but sub-contractors and key suppliers.

Pearson states that: “…(these) firms involve suppliers at an early stage in the project so as to acquire their expertise about design and procurement issues.” (A Pearson,”Chain Reaction”, Building, 12 March 1999 pp 54-55.)

A vision for better integration along the supply chain and between project phases

- The decision making processes, from project inception to completion, will be directed toward common purposes by communication between all parties.
4.3 Professional ethics and standards

**Neglect of professional ethics**

- Industry culture is self-serving and $ driven (little respect for the “triple bottom line”, i.e. financial, sustainability, community).
- Legalistic and adversarial contractual relationships are prevalent.
- Frequent use of litigation as remedial measure (and dependence on PI insurance solutions).

**Intervention Level**

- Attitude to professional ethics and standards has changed and become less valued and not consistently pursued.
- Preparedness by consultants to accept unreasonable briefs and then commit to delivering solely to commercial imperatives rather than a balance with professional accountability.
- Professional standards are afforded low priority and are diminishing.
- Processes for selection of professional services primarily based on cost criteria.
- Reluctant acceptance of a “you only get what you pay for” syndrome. Yet still seeking the higher level of service.
- Searching the documents for “loophole” opportunities to improve financial returns.
- Working relationships are not cooperative.

**Poor quality of design and documentation**

**Current state of professional ethics and standards**

Professional ethics and professional standards have become less valued and are afforded low priority as a result of commercial pressures and market competitiveness.

Consultants are tending to accept unreasonable briefs and then commit to delivering solely on commercial imperatives, rather than professional accountability.

Professional ethics are adequately defined by most industry bodies. However the term “appropriate standards” does not translate to an appropriate level of documentation to enable efficient construction of the project.

Relationships between client and consultant have become more contractual and adversarial, rather than cooperative. Most clients select a consultant on the low bid, whether or not appropriate services can be provided for that cost. A few informed clients have tried to address this but the outcomes have been disappointing.

The competitive low-bid environment corrodes professional ethics and professional standards among those operating in that environment.

Compromising ethics and standards allows underpricing of the necessary work to win the job. The consultant’s input is then limited by price, with an increasing likelihood of searching documents for “loophole” opportunities, and a legalistic and contractual approach to the professional relationship with the client.
How often do professional engineers and architects compromise the code of ethics of their professional bodies?

As members of the Institution Of Engineers, Australia, professional engineers are required to commit:

- to respect the inherent dignity of the individual
- to act on the basis of a well informed conscience, and
- to act in the interest of the community

Yet how often are the following Cardinal Principles of their Code of Ethics honoured by members of Engineers Australia:

- risk being managed in the interest of the community
- the responsibility of service to clients or employers
- practice being in accord with sustainability and environmental principles
- fairness in dealing with others
- relationships being on an open and informed basis
- knowledge being current to serve best the interests of the community, employers and clients
- awareness of the consequence of actions

Members of Engineers Australia in complying with their Code of Ethics should:

- “promote the principle of selection of consulting engineers by clients upon the basis of merit as well as fees, and should compete with other consulting engineers on the same basis”
- “should advise their clients or employers when they judge the project will not be viable, whether on the basis of commercial, technical, environmental or any other such risk…”
- “should not undertake professional work under terms and conditions that would compromise their ability to carry out their responsibilities in accordance with recognised professional standards.”

How much better would documentation be if these principles were universally applied?

A vision for professional ethics and standards

- All parties will behave ethically and fairly toward each other and the professional relationship between the parties will be clearly understood and valued.
- The consultant’s responsibility to act in the best interests of the parties without compromising the needs of the client, the community and the profession by providing independent and informed advice will be recognised through equitable remuneration.

Objectives for ethics and standards

- The prime focus of consultants will be satisfaction of community and client needs but with due recognition of operational and whole-of-life requirements and a fair and reasonable approach by all parties to all matters contractual.
- “Fair and reasonable behaviour” – a concept now being underwritten in Australian law courts – will be written into contract documents and accepted as a normal standard.
- Current codes of professional and ethical behaviour (Codes of Practice) – modified where necessary – must be clearly understood and effectively implemented by all parties.
- Continuing registration will be conditional on competent standards of performance and compliance with recognised Codes of Practice.
- On projects of significant scale and/or complexity all consultants will be accredited under recognised industry schemes.
- The availability and/or cost of Professional Indemnity insurance may be linked to conformity with the Codes of Practice and the bidding of adequate fees.
- Professional bodies will encourage innovation.
- Professional bodies will actively promote and encourage high standards of ethical and professional behaviour by all industry participants, and encourage formal training and mentoring for practitioners – including undergraduate, postgraduate and in-service courses.
- Concerns about inadequate performance in the terms of recognised Codes of Practice will be resolved with the relevant National Registration Board and/or the relevant professional body under a common commitment to ethical behaviour.
4.4 Selection strategy and bidding philosophy

**Price driven selection**

- Belief in economic policy that simple supply and demand fix price levels.
- Community culture in which low price = value for money, i.e. unsophisticated purchasers.
- Client view/culture that a fixed price agreed beforehand is needed to control project outcomes of cost and scope.

**Intervention Level**

- Selection of provider on a lowest bid basis.
- Procurement methods suitable for commodity purchasing are wrongly used to purchase sophisticated professional services.
- Misalignment in the project delivery equation (time=cost=quality=scope) occurs.
- Claims/variations are made.
- Cost and time overruns occur.
- Adversarial behaviours – poor relationships.
- Sub-optimal designs and compromised quality ("only get what you pay for").

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**Current selection strategies and bidding philosophies**

Selection criteria are not always appropriate. Selection of consultants is often driven more by price than the required level of service and expertise necessary for a successful outcome.

In a competitive environment, particularly where price is the critical factor in the assessment of the bids, a consultant cannot include more than what is required by the brief. This leads to a reading down of the brief to provide minimum service for minimum fee and consequent tension between the parties when the work done does not meet project objectives.

As Tilley (“Design and documentation deficiency and its impact on steel construction”) observed: “When design fees were reduced below the optimum level, project costs increased sharply due to increases in design
deficiency... Clients by their own actions were not getting the service their projects required, and this was leading to inefficiencies and additional project costs.”

In some cases, the briefing document is inadequate does not clearly indicate the required scope of work thus compounding the problem.

Selection strategies do not generally recognise that there are both definable and indefinable extents of work required of the consultant, and require the consultant to bid a fixed price for the unknown.

**As a consequence adversarial relationships develop, optimum design is not achieved, programs are not met and quality is compromised.**
A vision for effective bidding and selection strategies

- All service providers will be selected on the basis of value and competency - and will not be selected on the basis of lowest price alone.

This vision is supported by the work of Andi & Takayuki Minato ("Design documents quality in the Japanese construction industry: factors influencing and impacting the construction process") which contends that, "… The reduction in the level of design fees together with limited time made available to carry out the work can cause problems in the quality of design documents. Further, these problems have affected the efficiency of the construction process."

Bidding and selection objectives

- Buyers will:
  - have a sophisticated understanding of the term "value" which incorporates:
    - capability to deliver the required service to the nominated standard and within nominated time and cost constraints;
    - reliability and capability to perform in a non-adversarial manner;
    - social and environmental responsibility; and
    - whole-of-life cost implications.
  - understand the links and trade-offs between time, cost and quality;
  - appreciate that with professional services, they will usually get only what they pay for;
  - appreciate that many designs are prototypes so that procuring design and documentation services for infrastructure facilities differs from purchasing a commodity;
  - appreciate that an understanding of the owners’ business is of value to a designer and this is enhanced by development of long term relationships, that is, by repeat business;
  - encourage innovation by consultants and fairly compensate them for initiative;
  - recognise reputations in the industry and demonstrable evidence of innovation;

- understand the risk profile / risk allocation being between the parties;
  - acknowledge the skills required for the project and the capabilities required of design teams proposed by prospective designers;
  - have access to reliable and proven techniques for bid evaluation and contractual performance measurement.

- Procedures for selection of consultants will be based on assessment of value for the service offered (e.g. using Qualification Based Selection, ACEA or techniques practised in USA and Canada based on the Brooks Act).

The Brooks Act (1972) sets out US Federal Government policy requiring any Federal department, agency, or bureau seeking professional services of an architectural or engineering nature from any firm or individual to “… negotiate contracts for architectural and engineering services on the basis of demonstrated competence and qualification for the type of professional services required and at fair and reasonable prices.”

Specifically the agency head shall negotiate contract with the highest qualified firm … taking into account the estimated value of the services to be rendered, the scope, complexity, and professional nature thereof. Should the agency head be unable to negotiate a satisfactory contract with the firm considered the most of the most qualified … the agency head should then undertake negotiations with the second most qualified firm … (and so on).

More than 40 of the United States are already (2002) using Qualifications-Based Selection (QBS) of professional architectural and engineering services along the lines of the Brooks Act.

- Bid documents (proposals) will address all of the selection criteria and will contain sufficient information to support a value-based selection process.

- Trade practices legislation will recognise that competition for design services based on capability produces a more competitive and economical project in the long run than competition on price alone. There needs to be acceptance of non-prescriptive industry guidelines on fees.

- The opportunity to compete on capability, quality and service will be encouraged.

“Given that the work of architectural, engineering and surveying consultants is largely a ‘complex intellectual service’, many commissions are awarded on the basis of quality and price since it is recognised that you cannot get a quality service if just the lowest price is accepted.” (CQSA, 1992). ("Balancing fee and quality in two-envelope fee bidding", Derek S Drew, Sandy L Tang and Christine K Lui)
4.5 Risk management

Poor risk management

- Increased project complexity and greater societal expectations (e.g. environmental responsibility) provides greater risks.
- Project risks inadequately assessed and life-cycle implications not addressed by clients/developers.
- Lack of appreciation that risk assessment is a fundamental component of the project planning and design process – likelihood and consequence of project circumstances not adequately assessed.

Intervention Level

- Poor understanding of formal risk assessment and management processes.
- Identification and introduction of new risks not originally incorporated into the client brief.
- Adverse financial impact through need to revise documents.
- Inadequate communication/relationship with the client.
- Unpreparedness to allow time/cost for assessment of introduced (new) risks.
- Documentation does not totally deal with the mitigation of the assessed risks.
- Sub-optimal designs and comprised quality (“only get what you pay for”).

Risk assessment and management processes

- risks inherent in complex projects are not adequately identified, appreciated and/or fully assessed;
- the role of risk assessment in project planning and design processes is not fully recognised;
- formal risk assessment and management processes are poorly understood;
- there are a number of excellent risk management tools already in use in both private and public engineering and/or construction organisations.
A vision for risk management in the future

- Identification and analysis of all risks and uncertainty inherent in the project and its circumstances will form an integral part of project management.
- All parties will assess and manage risk and will commit to using competent processes for identifying, analysing and mitigating risks at all stages of the project.
- Fair and equitable risk management and allocation processes will play a significant role in coordination and integration along the supply chain.

Risk management objectives

- Project briefs will provide for risk assessment techniques to be applied early in all phases of a project, and for involvement of all parties along the supply chain at the appropriate stage.
- Adequate time, resources and funds will be provided to ensure that risk assessment and allocation is a key part of all project development, management, and design processes – including provision for mitigating risks throughout the construction process.
- Owners, consultants and constructors will be competent in the application of risk assessment and risk management techniques.
- Value engineering techniques will be implemented in determining appropriate measures for managing and mitigating project risks. Value engineering techniques control:
  - identifying the major elements of a project,
  - analysing the functions each project element performs,
  - brainstorming to develop design alternatives to perform those functions,
  - evaluating the alternatives to ensure they do not degrade the project,
  - assigning costs (including lifecycle cost) to the most promising alternatives, and
  - developing the most promising alternatives into acceptable recommendations.
- Specialist risk assessment staff will be engaged on all major and/or complex projects, or where project circumstances indicate that particular expertise is warranted.
- A coordinated and structured approach to risk management will be achieved through regular formal risk management meetings and will be assessed as part of regular project management reviews.
### 4.6 Design management

#### Absence of client design manager

- Evolving nature of economic/political markets forcing changes to procurement methods and design process (i.e. Fast Track, D&C).
- Increased demand and complexity of statutory regulation assessments and approvals requirements.
- Increased building complexity and increased number of expert specialised consultant’s advice required.
- Increased expectations to handle and process design/changes faster.

#### Intervention Level

- Lack of “qualified” client appointed design manager formulating, overseeing, project integrity and continuity.
- Unclear definition of scope and project briefs, risk management and accountability.
- Pressure to sign contracts early on ambiguous documents unclear allocation of roles and responsibilities.
- Lack of time available for constant and effective communication between parties (i.e. regular project meetings, etc.)
- Lack of funds for staff training, further research and interconnectivity with other disciplines.
- Lack of expertise/resources to check and co-ordinate all information on documents produced.

#### Poor quality of design and documentation

**The need for a qualified client-appointed design manager/coordinator**

There is a need not fully recognised by industry for an appropriately qualified and experienced professional to be appointed as the Owner’s design manager/coordinator with responsibility across all project phases from project initiation, planning and design, to construction and handover.

Greater complexity of projects, a much wider range of specialised professions, coupled with increased statutory requirements and approval steps have increased this need.

Economic and political factors often dictate fast tracking procurement methods (e.g. Design and Construct), but the project manager is neither assigned the full scope of this broad responsibility nor engaged for the feasibility stages of project design.

Consequently, briefs for engagement of professionals do not clearly define scope, risk management, accountability functions and responsibilities.

Time and cost pressures restrict the time necessary for review and negotiation of scope of professional services including their interface with all other components of project design, and often lead to pressure to sign-up on ambiguous documentation and unclear statements of roles and responsibilities.
A vision for the client’s design management

- Continuing client involvement in the design management and a strong relational ethic will be fully ingrained in industry culture.
- The client’s design manager/ coordinator will steer project design through all project phases from business case to commissioning; ensure alignment with project objectives; and ensure integration and coordination of the design effort with all parties through all stages of the project.

Objectives for effective design management

- Clients will be fully aware of the importance and benefits of the roles in their project team for Project, Design, and Construction Management.
- The client’s design manager will be an appropriately qualified and experienced professional.
- The client’s design manager will be the principal co-ordinator of specialist consultants and will maintain a consistent approach to obtaining project approvals and permits and to eliminating potential roadblocks.
- The design management team will be thoroughly briefed on the client’s functional requirements and budget, coordinating the various design and documentation professionals and the flow of information to ensure optimum and timely solutions.
- Group problem solving techniques involving all parties should include value management, value engineering, risk management and information management.
- Continuity of project management staff will be targeted from inception to commissioning – availability of the same staff to design and documentation consultants will be recognised as vital, with accountability and transparency across all disciplines maintained during all phases of the project.
4.7 Design process

Poorly documented designs

- Cyclic nature of the design process, from abstract to concrete concepts contrasting with the definite and fixed timing of contractual obligations.

- Increased sub-ordinances, statutory regulations, approvals and requirements.

- Increased complexity of technical construction, services and infrastructure interconnections.

- Increased speed, quantity and overload of information to be processed and produced.

- Increased availability of new IT tools, processes, products, materials, etc.

Intervention Level

- Lack of funds for the preparation of adequate design briefs, feasible programming, risk management and accountability at critical design stages.

- Lack of time for ongoing reviews and assessment of design decisions/evaluations of options that account for the cyclic nature of design process.

- Consulting environment averse to knowledge sharing necessary to synthesise decisions and information at every phase of the process.

- Lack of interconnectivity and efficient communication between all stakeholders to co-ordinate all documents.

- Lack of qualified staff and time available for checking and correlating all the information on all design documents.

Poor quality of design and documentation

Lack of funds for adequate design briefs, programming, risk management and accountability at critical design stages

Inadequate fees and tight, unrealistic programs, curtail the concept and schematic phases of the design. Thus the design concept may not be optimised, resulting in the client paying more than otherwise necessary.

At the other end of the design phase, detail design and design review may be similarly compromised

Computer-aided analysis, design and drafting have all led to an increase in productivity, but the need for communication and coordination has not diminished and still takes as long as it ever did.

This area of research is currently being addressed by a major international effort, called the International Alliance for Interoperability (IAI), which has developed a standard called the Industry Foundation Classes (IFC’s). This standard provides specifications for a set of standardised object definitions, which allows the transfer of information between software applications. “A Survey On The Impact Of Information Technology On the Canadian Architecture, Engineering And Construction Industry”, Hugues Rivard,
Assistant Professor, Concordia University, Canada

In addition, most projects are now more complex and are the subject of increased statutory regulation, governance and approval requirements.

Computer-aided analysis, design and drafting necessitate increased levels of checking due to the "black box" nature of much of the engineering, architectural, drafting, and quantity surveying software, which allows relatively inexperienced staff to carry out the work, and the use of "cut-and-paste" in CAD that results in many errors. CAD has also removed the opportunity for the progressive checking of drawings by both the drafter and the engineer. Despite the need for more checking, the current design environment often leads to less checking.

A vision for the design process

- The benefits of spending sufficient time and money in project planning and design will be widely understood and accepted by clients, owners and financiers – they will fully appreciate that an additional $1 spent in design optimisation has the potential to save $10 in construction and $100 in operating costs.

"93% of contractors indicated that design and documentation quality did influence the price submitted for a tender, while 75% of contractors indicated that it also had an influence on the time allowed for a project". ("Engineering Documentation Standards", Institute of Engineers Australia and AISS Special Issue, December 2000)

Objectives for the design process

Clients, owners and project financiers will:

- understand that design is an iterative process and an array of options needs to be identified, tested and costed before the optimal design is confirmed;
- understand that options need to be tested through value engineering processes to ensure that project objectives will be fully and efficiently satisfied;
- allow adequate and reasonable time for the design process to run its full course;
- appreciate that purchasing consulting services is not like purchasing commodities where quality is a constant and price can be negotiated;
- appreciate that making project cost savings by cutting costs in the design phase is illusory.

Designers will:

- draw the client’s attention to any need for analysis of alternative project design features and the potential benefits available;
- carefully check all designs for completeness and accuracy;
- ensure that experienced practitioners play a role in the documentation process;
- allow appropriate time frames to develop concept design, review and then carry out detailed design;
- take a pragmatic and appropriate attitude to development of a design that meets the buyers functional needs and is constructible.
### 4.8 Human resource capacity

#### Insufficient skills bank

- As Universities have become more reliant on self-funding, expensive courses like engineering are not well supported and student numbers are falling.

- Experienced industry people from the boom decades of the 60s and 70s are retiring, depleting the ranks of wisdom and knowledge and reducing the opportunities for mentoring.

- The industry funding structure constrained by input costs, provides financially unattractive career prospects for potential entrants.

- Cyclic nature of the industry leads to transient nature of the workforce, and low commitment to growing and retaining skills.

#### Intervention Level

- Adequate numbers of skilled and knowledgeable human resources are unavailable to the industry.

- Fee levels are not high enough to support genuine in-house on-the-job training and skill development.

- Robust competence level training for professionals and sub-professionals is not available.

- Sub-professionals have taken on “new” technology CAD skills at the expense of fundamental design capability leading to poor checking of designs.

#### Poor quality of design and documentation

#### Skill shortages and lower staffing levels

The construction industry is heavily dependent on development experience and retention of personnel having sophisticated experience, skills and knowledge.

However, universities are attracting fewer and fewer talented students to engineering and architecture courses. This, together with rapidly rising skill requirements and the retirement of ageing stalwarts of the industry, is leaving the industry in a depleted state to meet the challenges.

Further, due to the competitive low-fee focus in the industry, professional and para-professional personnel are poorly rewarded and underdeveloped due to low investment in training, skills, and knowledge development.

This issue is linked to declining fee levels and has a direct impact on the declining experience of staff; the main issues are as follows:

- low fee levels mean that staff training expenditure is reduced;

- apprenticeships for draftsmen have almost disappeared; relevant tertiary training is difficult to find and the courses offered are often limited in their relevance to the industry; and

- there has been a decline in the level of on-site training for both engineers and draftsmen. (“Engineers, documentation and litigation”, A Baigent, 2000).
As an inevitable result of inadequate numbers of skilled and knowledgeable resources, design and document quality are adversely affected and the community is suffering enormous losses.

“Findings from the interviews ……. revealed that a large portion of the rework costs experienced in the projects were attributable to the poor skill levels of the clients’ project manager, the design team and subcontractors – training and skill development were not issues that the consultants addressed, primarily because of the associated costs involved - the lack of training and skill development with technology applications such as CAD adversely affected motivation levels amongst those employees working on projects. (“Determining the causal structure of rework influences in construction”, P Love et al, 1997)

“Both contractors and designers indicated the increasing use of junior and inexperienced staff to carry out the design function - one of the major concerns of clients, contractors and other consultants (in relation to documentation quality) is the apparent lack of knowledge of designers in relation to practical building and construction methods (constructability), not only when they are commencing their careers, but also throughout their careers” (“Design and documentation deficiency and its impact on steel construction” - (P Tilley, 1998)

A vision for human resources

- Design and documentation personnel available across all disciplines will be well trained, experienced and competent, and will be properly recognised and rewarded for the skills and responsibilities required.
- Continuing professional development (CPD) will be accepted as an essential requirement for ensuring that staff maintain up-to-date qualifications and competency standards.
- Mentoring will be strongly encouraged and recognised as an essential practice to make sure that relevant experience is passed on to less experienced personnel.

Drivers of Change (include) commitment to people: this means not only decent site conditions, fair wages and care for the health and safety of the work force; it means a commitment to training and development of committed and highly capable managers and supervisors; it also means respect for all participants in the process, involving everyone in sustained improvement and learning, and a no-blame culture based on mutual interdependence and trust.” (“Rethinking Construction”, The Egan Report, 1998)

Objectives for human resources

- The level of skill required for the industry will be recognised by clients and rewarded appropriately.
- Undergraduate and graduate programs at colleges and universities will closer reflect the needs of the industry and will be developed in close co-operation with industry associations and relevant client groups.
- Opportunities will exist for trainees to combine work and study through part-time courses.
- Less experienced personnel and trainees will be included in design teams as an essential practice for developing their experience and competence, and for ensuring the long-term viability of the industry.
- Professional bodies will implement strategies and programs for professional development of their members (design professionals etc).
- Continuing professional development will be mandatory, well planned and assessable as a condition for continuing involvement in the industry.
- Research and development activities will be more widely available, incorporated into the time allocation for professional staff and supported by appropriate fee levels.
- All professional and technical support staff will be comfortable working with IT/CAD and will be familiar with 3D and 4D modelling of their designs.
### 4.9 Technology

#### Ineffective use of technology

- **Unstoppable technological evolution**
- **Pace of change – designers cannot know pros and cons of new products, but can’t afford to ignore them.**
- **Society’s expectation that technology will significantly reduce the cost and delivery time of goods and services.**

#### Intervention Level

- **Inadequate use of CAD/technology for design purposes.**
  - Acceptance of the output of computer packages without sufficient knowledge of the concepts on which they are based.
  - Reluctance to change completed drawings to improve the design when looked at by experienced personnel.
  - Incompatibility of systems used by different design systems.
  - CAD drawing errors due to the nature of the process (i.e. lack of checking, quality of checking).

#### Poor quality of design and documentation

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### The status of CAD and the technology revolution

The evolution of technology is rightly unstoppable and inevitable.

CAD promised the paperless office, yet we now consume more paper than ever before. Technology promised to improve efficiency, yet now there is so much information that there is little time to sort it, let alone prioritise and check it.

Computers have helped to produce more documents at greater speed, but they have also created the chances of producing uncorrelated, ‘black box’ solutions which are either incorrect, ambiguous or inappropriate for the situation. The possibility of errors has multiplied and the quality of design and documentation has declined.

This is largely due to the rise of “machine operators” who are not necessarily competent designers but know how to operate the complex software albeit not understanding nor delivering design quality. (“Engineers, Documentation and Litigation” (clause 3.5 – Inappropriate reliance on technology), Dr Andrew Baigent in the AISC – IE Aust Special Issue Steel Construction, Vol 34, No 4, Dec 2000)

Design by computer distances the designer from the real issues and discourages independent checking by knowledgeable practitioners and cross-checking with guides, standards and other relevant documents. These result in design that is not holistic but rather a design segmented into layers lacking integration and not achieving design objectives. Unique and efficient design solutions must be by a qualified and experienced designer.

Reliance on computer generated solutions without designer judgements will lead to solutions of a prescribed nature that invariably produce inappropriate solutions.
So far, technology has not fulfilled its promise to promote greater efficiency and better design but rather has contributed to poor quality of project design documentation.

**Objectives for managing technology**

- Projects will be supported with integrated virtual models that will be used throughout the design and operation phases.
- Communication between all parties will be effective, well tried and very open in nature.
- Design and communications software, hardware and computing networks used throughout the industry will be thoroughly compatible.
- Accurate and accountable technology and communication will be available - mitigating risk and ensuring a higher standard of documentation.
- Sharing project information via sophisticated computer models will provide more accurate data on the status of the documentation at any given time.
- All designers will be able to share digital information models in a single database - constantly updating information rather than each relying on their own systems.
- Design documentation will be directly linkable to component manufacturing processes and on-site construction and be available post-construction for facility management systems.
- Users of sophisticated systems will be competent design and/or documentation professionals first and foremost, with a good understanding of the principles of design, merely using technology as the most appropriate method to achieve their purpose.
- Consultants will be aware of the need to ensure wisdom and expertise is incorporated throughout the design process - ways of progressively reviewing and overseeing the design by skilled practitioners are an integral part of the overall process (e.g. for incorporating constructability features and owner input).

**A vision for managing technology in the future**

- **The building and construction industry** will embrace technological change – eager to capitalise on the potential that technology offers for achieving challenging objectives, continuous improvement and long-term viability.
- **The role of technology in the delivery of projects** will be thoroughly understood and encouraged by all stakeholders.
- **Accredited software packages** will satisfy the following objectives:
  - assist in the delivery of best quality design and documentation;
  - facilitate system compatibility and interchange;
  - include training modules to maximise efficiency and effectiveness of their application.
4.10 Communication

Lack of open communication

- Adversarial climate and blame culture make communications more difficult.

- Modern technology (email and mobile phones) improves contact ability and interferes with ability of individuals to dictate their own priorities and agenda setting.

Intervention Level

- Lack of appreciation of the essential nature of open communications.

- Closed/secretive behaviours (i.e. consciously hiding information) – self-interest rather than best-for-project.

- Time and cost pressures restrict available time for talking/communicating (and building relationships).

- Documentation not developed/presented in format/terms than can be interpreted by site workers.

- Inadequate involvement of experienced designers in design team.

- Lack of coordination between technical disciplines.

Poor quality of design and documentation

Lack of appreciation of essential nature of open communications

Communication in the building and construction industry is constrained as a result of the adversarial climate and the blame culture this climate has engendered. This is one of the consequences of extremely tight time and cost pressures in a very competitive environment.

“The reluctance to interact is also fuelled by the perception that adversarial relationships must exist as they historically have between designers and contractors – adversarial relationships arise when parties blame each other, even when it is impossible to assign blame to one party exclusively.” ("Leveraging specialty-contractor knowledge in design-build organisations", N Gil et al, 2001)

Closed (and even secretive) behaviours with a focus on self interest (at the expense of best-for-project interests) are taking precedence over the essential requirements for open communication between all parties in the very complex project environment.

Time and cost pressures severely restrict the time available for oral communications – so essential for making sure that issues are identified, thoroughly analysed, talked through and the outcome understood in the same terms by all parties.

Improved contactability provided by modern devices (mobile phones and email) interferes with the ability of individuals to define and implement agreed programs of action or to progress priority activities.

“Poor communication and inadequacies in interactions between designers and other parties contribute to rework in Australian projects.” (CIDA, 1995)
“(Participants in project meetings) fail to recognise explicitly what needs to be communicated and when.” (“Leveraging specialty-contractor knowledge in design-build organisations”, (N Gil et al, 2001)

“The manner by which drawing files are being sent appears to be so uncontrolled as to be bordering on dangerous.” “Engineering Documentation Standards”. (P Cacciardi, 2000)

A vision for open communication

- It will be accepted that the primary ingredient of successful project delivery is people working cooperatively together, sharing the same vision and objectives for the project.
- The contracting arrangement will be framed around goodwill and fair dealing in an open communication environment.

"Clear communications through oral and visual means will be the backbone of good project relationships.” (“Architectural management – an evolving field”, S Emmitt, 1999)

Objectives for improving communication:

- All project documentation will be easily intelligible to those parties along the supply chain who need to use it.
- For each project a communication plan will be established at the outset defining the roles of the parties and how their work will be facilitated by open communications.
- The effectiveness of project communication plans will be assessed as part of regular project management reviews.
- Face-to-face communications will be the basis on which common understanding and good relationships are developed and fostered.
- Information technology will be the tool used to improve and simplify communication rather than a means of merely transmitting ever increasing quantities of information at ever increasing speeds.
- Undergraduate, post-graduate and in-service training programs for industry professionals will include training in communication and relationship building techniques.
Section 5: Recommendations for Industry Change

Setting benchmarks – A vision for the future

To assist in identifying solutions for the various causes of the problem the Task Force has defined a vision for the future of project design documentation:

- When benchmarked against similar industries in leading OECD countries, the building and construction industry will be highly efficient and competitive, in a large part due to the care and skill applied to the development of project design documentation.
- Project designs will be characterised as innovative, fit for purpose and constructible because of the skill, knowledge and advanced technology that is applied.
- Documentation will fulfil the owner’s requirements, and be thorough, accurate, unambiguous, and easily communicated across the full spectrum of the construction industry’s workforce.
- An open learning environment facilitated by open communications will exist in the industry.
- Participants in all sectors (owners, clients, project managers, designers, detailers, constructors, suppliers and users) will:
  - be acutely aware of the need to maintain high standards of design and documentation, and
  - have a full understanding of the factors that bring about best standards of design and documentation.
- The industry at all levels will embrace the opportunities provided by information technology and will lead the way in the development and adoption of compatible systems and software that better support the needs of all parties.
- Sharing of information by open communication, consistency, and continuous improvement will be hallmarks of the culture of the industry – this will be facilitated by an awareness of the need to build skills and knowledge in all areas including project documentation and management.
- This culture will support fair dealing and operate in a spirit of good faith. Fair and equitable remuneration will be available for all parties. Judicious use will be made of industry related legislation and regulation supported by codes of practice so as to narrow the performance band.

A “no blame” approach to the task

<table>
<thead>
<tr>
<th>Industry Participants</th>
<th>Part to Play</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Owners</td>
<td>• provision of project briefs&lt;br&gt;• consultant capability and value ahead of lowest fees&lt;br&gt;• effective risk management</td>
</tr>
<tr>
<td>Professional Consultants</td>
<td>• competent deliverables ahead of the lowest fee&lt;br&gt;• professional ethics&lt;br&gt;• recruitment and on-the-job training</td>
</tr>
<tr>
<td>Constructors and Suppliers</td>
<td>• competent performance ahead of the low bid&lt;br&gt;• cooperation ahead of adversarial behaviour&lt;br&gt;• recruitment and on the job training</td>
</tr>
<tr>
<td>Financers and Lawyers</td>
<td>• a fair contract between equals rather than a tight (outdated) master servant contract&lt;br&gt;• cooperative approaches rather than adversarial</td>
</tr>
</tbody>
</table>

In a spirit of cooperation and communication across the industry, the issues, the root causes and the recommendations are set out in a no-blame non-adversarial manner. Nevertheless, certain sectors of the industry will need to take an active part in resolving a number of the issues:

In developing solutions to the 10 root causes set out in Section 4, two other factors influenced the approach of the Task Force:

- Firstly, there is significant overlapping of, and interaction between, the effects of each of these individual root causes.
- Secondly, several remedial activities would be beneficial in the resolution of all or most of the 10 individual root causes, such as:

1. Make sure that the whole of the industry and all of its players are fully aware of the current situation, the consequent waste of resources, and an increasing difficulty with the “do nothing approach”.
2. Promote the need for a change in culture, across the industry.
3. Reinforce existing codes of practice, guidelines for industry, and legislation and regulation (modifying where necessary and in some cases assembling new).
4. Strengthen the requirements for continuing professional development, tertiary training and in-service/on the job training.
5. Reinstate equitable remuneration for professional design services within a value for money equation.
6. Improve project delivery across the board.
To address the root causes, the Task Force has grouped its recommendations around four major categories:

5.1 **Project briefs – including some discussion on:**
- Risk assessment and allocation
- Communication practices

5.2 **Bidding philosophy and a selection strategy for consulting services**

5.3 **Project delivery – including:**
- Professional ethics, standards and accountability in business practices
- Risk management
- A client-appointed overall design manager
- Design process – what is required to optimise designs and provide quality documentation
- Adequate human resources
- Adequate and effective use of technology, e.g. CAD
- Effective communication practices

5.4 **Implementation strategy**
- Vision for the construction industry
- Building stakeholder support participation
- Managing the task
- Communicating and marketing the message
- Changing the industry culture and modifying behaviour
- Continuous improvement
- Everyone to play a part
Section 5: Recommendations for Industry Change

5.1 Project briefs

Root cause: Poor project briefs based on unrealistic expectations

Guiding principles

(*Refer to section 4.1 for the Task Force’s vision and objectives for project briefs)

The purpose of the project brief is to define, limit and allocate the uncertainty and risk associated with the project.

The project brief stage is an important and separate stage in the project procurement process for which the client has primary responsibility.

The project brief should be completed before the preparation of the consultant services brief.

An adequate brief

The subject of inadequate briefs was tackled by the Australian Construction Industry Forum and Australian Procurement and Construction Council Inc. in 2002. The aim was to identify the key issues in the apparent decline in project documentation quality over recent decades for government as the client and buyer of services and the construction industry as the seller and supplier of services. The report, “Improving Project Documentation – A Guide to the Current Practice” (March 2002), sets out five sets of principles and protocols. While recognising the merits of this report, we believe a broader document placing the whole project initiation stage in context is required. We reviewed CIDA’s “Construction Industry Project Initiation Guide for Project Sponsors, Clients and Owners” (CIDA 1994) for this purpose.

Briefs have been addressed by the Task Force based on a review of current practices and definitions of what briefs are, for example as defined in the above ACIF/APCC document.

The Project Initiation Guide may well be sufficient to set the norm for client project managers and professionals to develop effective project briefs. If so, our industry needs to promote understanding and widespread use of it.

The CIDA Project Initiation Guide recognises a three-step process in the formation of a project brief:

1. The concept stage evaluation brief:
   - To identify constraints
   - To describe a range of options
   - To select a shortlist based on analysis by functions/use; cost/benefit

2. The definition stage brief containing:
   - A description of the preferred option
   - Cost targets
   - Time requirements
   - Quality considerations
   - Redefinition of the functional, physical and financial constraints and objectives for the project

3. The project delivery brief which is expected to cover:
   - The enterprise objectives for the project
   - The functional objectives – what the project must do
   - The functional constraints
   - A summary of the feasibility and risk analyses
   - Details of planning approvals
   - The project implementation plan, actions and schedules
   - The procurement plan
   - A cost plan
   - The project documentation, description and illustrative definition
## Recommended Solutions

<table>
<thead>
<tr>
<th>Standards for project briefs</th>
<th>Actions</th>
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</thead>
<tbody>
<tr>
<td>Develop comprehensive, succinct benchmark standards for project briefs.</td>
<td>• Review and establish industry based guidelines/protocols for effective briefs, including checklists and quality case study examples.</td>
</tr>
<tr>
<td>Adopt and/or modify a set of protocols acceptable across industry for the development of project briefs e.g. the 2002 APCC/ACIF or the 1994 CIDA or the current Capital Works Management Framework (DPW – Qld).</td>
<td>• Engage Government agencies, significant private sector clients and industry associations (e.g. Property Council) as stakeholders in the process.</td>
</tr>
<tr>
<td>Establish clear definition of terms.</td>
<td>• Consider engagement of professional assistance at this time.</td>
</tr>
</tbody>
</table>

### Client and industry awareness of project briefs

Clients need to be made aware of the benefits of, and flow-on from, effective project briefs – particularly with project owners who have most to gain (or lose). This is particularly relevant for one-off clients.

- Present industry seminars, forums and training sessions on the benefits of developing briefs, and the skills required to develop them, with topics such as:
  - the use of independent consultants in the preparation of the brief where the client lacks the skill or experience in-house
  - proper scoping, which is essential for accurate planning
  - conducting general awareness program on effective Project Briefs and Consultants’ Services Briefs
  - clear client objectives and key drivers for the project being articulated to allow all service providers to respond to the true project goals
  - a comprehensive brief flowing on into effective risk management
  - including a Recommended Project Cost Plan and Project Master Program in the project brief
  - importance of identifying site restraints and existing infrastructure and services

- Gain commitment of professional associations, client bodies, property council and consultants teams to the importance of ensuring an adequate brief is completed before commencement of a formal commission.

- Encourage professional service providers to include finalisation and sign-off of brief as part of Quality Plan.
<table>
<thead>
<tr>
<th><strong>Recommended Solutions (cont)</strong></th>
<th><strong>Actions (cont)</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Specialised expertise</strong></td>
<td>• Engage specialist external consultants where the client does not have the necessary expertise in-house.</td>
</tr>
<tr>
<td>Successful brief preparation requires specialist expertise and experience, including, technical services, budgeting and programming.</td>
<td>• Upskill consultants in principles of brief preparation to assist the client when undertaking pre-design and development of the project brief. This may need to be considered a separate commission with different responsibilities, fees and provided by skilled practitioners in that field.</td>
</tr>
<tr>
<td></td>
<td>• Encourage participation of the client within an interactive communication process through the appointment of a Client Design Manager / Coordinator.</td>
</tr>
<tr>
<td><strong>Link to Professional Indemnity (PI) Insurance</strong></td>
<td>• Further communication with PI insurance providers to be undertaken to discuss this issue.</td>
</tr>
<tr>
<td>Encourage PI insurance providers to recognise in making PI insurance available and/or setting a premium that good briefs diminish risk and enhance the cooperative resolution of difficulties, and as a consequence reward those who apply good briefing practices.</td>
<td>• Establish industry accepted requirements for a project brief to be prepared prior to project initiation, which may form the basis of an agreement between the professional associations and the Insurance Council or Providers.</td>
</tr>
</tbody>
</table>
5.2 Bidding philosophy and a selection strategy for consulting services

**Root cause:** Selection of consultants on a lowest bid basis.

**Guiding principles**

("Refer to section 4.4 for the Task Force’s vision and objectives for consultant selection")

The purpose of a selection strategy is to identify and engage, on equitable terms, the most appropriate consultant(s) available, consistent with the scale, complexity and urgency of the project under consideration.

Appropriate remuneration is required to allow professionals to provide independent and informed advice with the focus on the extent and quality of service. A low-cost competitive bid may give savings of 2% to 3% on fees, but lead to project cost overruns of 10% to 15% or more, due to lower standards of project documentation.

The Task Force’s research shows:

- There needs to be a better understanding by clients of the concept of “value” where a greater investment in the early stages of project development (pre-design briefing, design / documentation) will generally result in a much improved building outcome in terms of both capital cost and facilities management/ lifecycle costs.

- Consultancy commissions should take account of the consultant’s capability, resources, reputation, relevant experience and design methodology together with the ability to deliver on functional requirements, time and budget expectations.

It is a basic tenet that efficient and competent application of the principles of competition to the whole of the construction industry requires far more emphasis on the capability and competence of the bidder with less emphasis on lowest price, and that this will result in a more competitive and competent industry with sufficient skills to meet the community’s expectations.

This is the basis of Qualification Based Selection (QBS) widely used by value-conscious owners.

Christadoulou et al (Qualifications Based Selection of Professional A/E Services) (2004) demonstrate that:

- Architectural and engineering services are highly specialised and do not lend themselves to the lowest-responsible-bidder procedure.

- QBS provides lowest lifecycle costs.

Opponents of QBS object to the process, claiming:

- Since QBS does not by definition consider price as a decider, the owner does not get the best price for the project.

- QBS is based on subjective criteria (qualifications) and not objective criteria (price) and thus it is not accurate.

- QBS limits approval of possible proponents to a select few thus favouring older established firms over new firms. Since the firm’s experience and expertise become the most important factors in the selection process, the critics ask how a firm can prove its qualifications without having the experience. How can a firm acquire experience if QBS limits the firm’s chances of obtaining contracts?

Christadoulou et al quote a case study by the American Institute of Architects comparing the selection processes for public projects in the states of Maryland and Florida. Florida emphasises technical qualifications followed by negotiation of a “fair and reasonable” fee using the QBS format, whereas in Maryland price was the dominant factor, 83% of projects going to the low bidder.

A comparison of projects in these two states showed that the QBS selection method used in Florida appeared to result in about one-half the cost of selection and design and about one-half the administration cost of projects in Maryland, while delivering projects in about three-quarters of the time.

For complex projects, very large contracts and projects where the documentation is immature, relationship contract formats are favoured where the emphasis is no longer on impractical low fees. Examples are alliances, partnering and various forms of incentive based contracts that involve negotiated fee structures where risks of cost over-runs and under-runs are shared between the parties on an agreed, equitable basis.
### Recommended Solutions

<table>
<thead>
<tr>
<th>Selection criteria</th>
<th>Actions</th>
</tr>
</thead>
</table>
| Promote consultancy selection tools that recognise qualifications of the proponents. | - Promote consultant selection criteria that take into account:  
  - current workload  
  - available resources  
  - past commission performance  
  - amount of repeat business  
  - experience on similar projects  
  - ability to meet the design /documentation program  
  - preferred secondary consultants  
  - likely fee position  
  - the ability to work in cooperation with the client, the other consultants and the project team |

#### Selection on value for money ("Qualifications Based Selection")

Each client organisation and each industry body should establish and adopt consultancy selection tools based on value for money, such as "Qualifications Based Selection". This may involve preparing new, or modifying existing guidelines and/or checklists on bidding and selection principles based on value selection; equitable risk allocation; whole of life costs; and establishing and weighting selection criteria.

**Existing practices**

Some other existing practices that go part way towards taking the focus off the low price bid as the sole or main selection criteria include:

- **a)** the two-envelope system
- **b)** discarding the significantly lower – say 10% lower – bid, or focusing on the mean or median priced bid
- **c)** negotiating on the basis of repeat business based on previous success

**Ethical selection**

- The industry must demonstrate to clients by means of industry seminars and/or direct contact that insufficient fees and premature commitment of work will increase the probability of inadequate documentation and significant contractual claims, whereas purchasing infrastructure designs on the basis of value, whole of life costs, and equitable risk allocation principles rather than price leads to a lower project implementation cost.

- Further, an accurate scope of services is required as part of the bid documents.

- Document case studies of projects where appropriate procurement strategies have led to highly satisfactory outcomes.

- The QBS approach may in time be introduced into legislation at least for Government procurement, as it has been elsewhere.

- Encourage selection assessment practices that are open, ethical and transparent.
### Other Avenues to be explored (Note 1)

<table>
<thead>
<tr>
<th>The Brooks Act (USA – 1972) (refer also page 21)</th>
<th>Actions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initiate a discussion with government on The Brooks Act which sets out US Federal Government policy requiring any Federal department, agency, or bureau seeking professional services of an architectural or engineering nature from any firm or individual to “… negotiate contracts for architectural and engineering services on the basis of demonstrated competence and qualification for the type of professional services required and at fair and reasonable prices.”</td>
<td></td>
</tr>
<tr>
<td>Note: The Queensland Department of Public Works does not support this recommendation because the Queensland State Purchasing Policy already permits the use of value selection of professional and construction services.</td>
<td></td>
</tr>
<tr>
<td><strong>Fee guidelines</strong></td>
<td></td>
</tr>
<tr>
<td>A solution may be to adopt recommended guidelines on how to calculate fees, formulated in consultation with all relevant stakeholders, and to switch the emphasis in the selection of a professional consultant from price to value, capability and experience. Research shows that competition based on value, capability and experience leads to a more price-competitive total project cost, but the concept of recommended components of a fee will require ratification by the Australian Competition and Consumers Commission (ACCC).</td>
<td></td>
</tr>
<tr>
<td><strong>An “AAA” type rating scheme</strong></td>
<td></td>
</tr>
<tr>
<td>Note: This proposal was not universally accepted and/or thought practical by the industry personnel who provided feedback on the draft report.</td>
<td></td>
</tr>
<tr>
<td>• Explore the issues and constraints associated with the introduction of State Government legislation (similar to the Brooks Act – which has been adopted by 40 of the United States) to mandate the use of value selection procedures for professional services for Queensland Departments and Government Owned Corporations (GOC’s) as an example to industry, and for the benefit of the taxpaying public.</td>
<td></td>
</tr>
<tr>
<td>• Recommended fee scales and guides such as the RAIA’s have generally been observed in the breach. A recent decision by the ACCC requires the RAIA to withdraw its Guide which is both out-of-date and based on a very small sample response to a fee survey some years ago.</td>
<td></td>
</tr>
<tr>
<td>• The RAIA has decided to view this positively and will produce a guideline on the calculation of fees based on cost and time records and measured overheads. This is part of an education process of members which has as its goal the achievement of incomes and profit levels which ensures practices are sustainable.</td>
<td></td>
</tr>
<tr>
<td>• The Queensland Government in a recent review of its State Purchasing Policy incorporated a review system which evaluates the performance of consultants and allows the possibility of rewarding good service by increasing the number or frequency of opportunities for work for those whose performance is rated at a higher level.</td>
<td></td>
</tr>
</tbody>
</table>
Note 1:
Existing legislation relevant to this and other issues

Legislative and regulatory actions are required to protect the community. The existing Queensland legislation that impacts on the provision of construction documentation and design includes:

- **Professional Engineers Act 2002 (Qld)** – protects the public by ensuring engineering services are provided by registered engineers in a professional and competent way; maintains public confidence in the standard of services provided by them; and upholds the standards of practice of registered professional engineers.

- **Professional Standards Act 2004 (Qld)** – enables the creation of schemes to limit the civil liability of professional operators; helps in improving occupational standards of professionals; and protects consumers of services provided by professionals and others.

- **Building Act 1975 (Qld)** – creates standard laws concerning the erection of buildings and other structures; provides for building certification; and the standard of documentation is under S14B-14D under the Standard Building Regulations.

- **Queensland Building Services Authority Act 1991 (Qld)** – regulates the building industry (1) to ensure the maintenance of proper industry standards and (2) to achieve a reasonable balance between the interests of builders and consumers. The Act establishes the Queensland Building Services Authority with responsibly to license people who perform building work, provide dispute resolution services, administer a statutory insurance scheme and administer the disciplinary provisions of the Act.

- **Architects Act 2002** – protects the public by ensuring architectural services of an architect are provided in a professional and competent way; maintains public confidence in the standard of service provided by architects; and upholds the standard of practice of architects.

Codes of practice have now been gazetted under the Professional Engineers Act (2002) and Architects Act 2002 being respectively “Code of Practice for registered Professional Engineers of Queensland” in June 2005 and “Board of Architects of Queensland Code of Practice” in November 2004.

Several of the above listed acts, however, could be further strengthened by the addition of a reference to what constitutes adequate project documentation.

However there is some industry criticism of the effect of continual changes in the law on the industry and some stability of legislation is necessary.
5.3 Project delivery

Arresting and/or reversing the declining standard of project documentation as a deliverable under professional consultants’ commissions will require a strong focus on the following aspects of the project delivery system:

1) a renewed commitment by all professional consultants and their clients to – and if necessary mandating under the Professional Engineers Act and the Architects Act – existing industry Codes of Practice, a renewed commitment to restoration of professional ethics as a rightful determinant of professional behaviour, a commitment to raising of professional standards, and a commitment to acceptance by their professions of accountability in everyday operations and in business practices.

2) acceptance by both client and consultant of the need to identify risk and opportunity and to allocate them according to the proper principles of risk management.

3) restoration or establishment of the role of an overall client-appointed design manager or coordinator on contracts of any significance to monitor the performance throughout the design process.

4) renewed appreciation of what is required to optimise designs, and provide documentation of a sufficient quality during the design process.

5) addressing by the whole industry including professional organisations, academia and the government, on the current shortage of professional skills and human resources – whilst ensuring adequate standards remain in place.

6) addressing current inadequacies and/or ineffective use of modern technologies such as CAD drafting, ability of computer systems to talk to each other along the supply chain, and computer-based communications.

7) rationalisation of communication rules and practices.

8) developing process control appropriate to project size and complexity.

Legislative and regulatory support

The existing Queensland legislation that impacts on the provision of project design documentation includes:

- Professional Engineers Act 2002 (Qld)
- Professional Standards Act 2004 (Qld)
- Building Act 1975 (Qld)
- Architects Act (2002)

However, a wider debate among industry participants, together with the government, is required on:

- The effectiveness and implementation of these acts, and
- The potential for an act such as the Brooks Act (1972, USA) or regulations such as through the State Purchasing Policy to encourage emphasis – at least for public/government projects – on Qualifications Based Selection rather than on selection mainly on price.

For example, in 1998 the Queensland chapters of the Master Builders Association, the Australian Institute of Quantity Surveyors, the Royal Australian Institute of Architects, and the Building Industry Specialist Contractors Organisation have agreed that the principal/developer ought to prepare full “Bills of Quantities” for all building projects in excess of $1 million. Legislation or regulation under the Building Act 1975 or other appropriate act could set this good practice into stone.
5.3.1 Professional ethics, standards and accountability

**Root cause:** *Professional ethics and standards have been devalued, and are not consistently adhered to.*

Renewed focus and understanding of professional ethics and standards are required as part of a new culture for the industry. Participants need to be more equitable and fair minded in their outlook and be truly accountable for their professional behaviour. The following solutions and actions are suggested for consideration:

<table>
<thead>
<tr>
<th>Recommended Solutions</th>
<th>Actions</th>
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<tbody>
<tr>
<td><strong>Obligations under Codes of Practice</strong> Ethics are essentially related to personal professional standards An appreciation is required of the difference between a code of conduct on one hand, and standards for service provision on the other.</td>
<td>• Promote Codes of Practice within industry, within client groups and to the community at large, emphasising minimum standards and encouraging participants to exceed minimum standards. • Promote, both within industry and general community, the importance of adherence to ethical obligations by professionals and steps taken by the industry to ensure such adherence. • Promote cooperative relationships with clients, as opposed to adversarial or legalistic ones.</td>
</tr>
<tr>
<td><strong>Disciplining breaches</strong></td>
<td>• Support the professional associations in applying disciplinary mechanisms for professionals who fail to adhere to their Code of Ethics.</td>
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</table>

5.3.2 Risk assessment

**Root cause:** *Poor understanding of risk identification, allocation and management process and lack of risk management knowledge and skills*

Effective risk management is crucial for good project outcomes. It should be standard practice on all projects to formally identify all project risks, to evaluate these risks and to institute appropriate strategies to monitor and manage impacts. A critical part of the management process is the fair and equitable allocation of the risks to contracting parties. The following solutions and actions are suggested for consideration:

The Australian Standard on Risk Management AS/ NZS 4360:2004 sets out an excellent outline of the risk management process – a process that ought to be adopted by every member of the building and construction industry.

An outline of this process follows:

- a) communicate and consult with all stakeholders inside and outside the organisation throughout the process
- b) define the project, the time and location and the nature of the decisions that have to be made, identifying any scoping or framing studies needed, and the roles and responsibilities of the participants in the risk management process
- c) establish the external, internal and risk management context and the criteria against which risk will be evaluated. The external context may include, for example:
  - the business, social, regulatory, cultural, competitive, financial and political environment
  - strengths, weaknesses, opportunities and threats
  - perceptions and values of external stakeholders, and
  - key business drivers
Key areas for the internal context include:

- Culture
- Members of the organisation
- The organisation’s structure
- Capabilities of the people, systems, processes, and capital, and
- The organisation’s goals and strategies

The criteria to evaluate risk may be based on:

- Operational, technical, financial, legal, social, environmental, humanitarian, or other factors

d) identifying the risks including what can happen, where, when, why, and how events could prevent, degrade, delay, or enhance the project objectives, and whether or not these risks are under the organisation’s control. Tools and techniques include:

- checklists, judgments based on experience and records, flowcharts, brainstorming, public consultation, systems analysis, scenario analysis, and systems engineering

e) analyse the risks quantitatively or quantitatively with sensitivity analyses, analyse the existing controls, and the consequences and the likelihood and hence the level of risk

f) evaluate the risks considering the balance between potential benefits and adverse outcomes, including the sources of risk, their positive immediate consequences and the likelihood of them occurring

g) develop and implement specific cost-effective strategies for increasing the benefits and reducing the costs of opportunities and risks by:

- seeking opportunities and avoiding risks
- altering the likelihood of opportunities/risk
- altering the consequences to enhance benefits and reduce costs
- sharing the opportunities/risk, and
- retaining the opportunities/risk

A combination of risk treatment options may be adapted for example, effectiveness of contracts with appropriate insurance and other risk financing.

h) monitor all steps of the risk management process for continuous improvement and to allow for changing circumstances

i) keep records, communicate and consult at every stage, bearing in mind:

- the legal and business needs for records
- the cost of creating maintaining records, and
- the benefits of using information

Risk management planning involves:

1. developing a risk management plan
2. obtaining the support of senior management
3. developing and communicating the risk management policy
4. establishing accountability and authority
5. customising the process the organisation’s policies and culture, and
6. ensuring adequate resources

(Reference: AS/NZS 4360:2004 Risk Management Standards Australia and Standards New Zealand)
<table>
<thead>
<tr>
<th><strong>Recommended Solutions</strong></th>
<th><strong>Actions</strong></th>
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</thead>
</table>
| **Risk management systems** | • Research and develop risk management frameworks (including risk templates and software useful for all levels of the construction industry – from project inception to completion).  
• Adopt these frameworks as benchmarks in accordance with the Australian Standard 4360:2004 – Risk Management. (*refer Note 1)  
• Develop standard documentation formats such as Natspec in conjunction with providers, contractors and the Insurance Council of Australia. |
| **Poor documentation risks** | • Ensure that risk management plans (including poor documentation risk) are routinely used in the development stages of projects.  
• Introduce risk awareness/identification by all stakeholders in a project as a basic management process (a simple management matrix).  
• Have risk management processes taken into account for particular indemnity requirements, professional registration and licensing.  
• Encourage clients to Implement Independent Risk Auditing of project documentation. |
| **Industry awareness** | • Raise industry awareness of the nature and benefits of effective risk management. |
| **Learning program** | • Integrate risk management as part of all learning institution programs for the building and construction and allied industries. |
### 5.3.3 Lack of integration across the supply chain, and lack of cooperation and communication between the parties

**Root cause:** Poor integration along the supply chain, *i.e. between project phases*

Successful delivery of complex projects with many stakeholders in an uncertain environment requires the integration of the planning, design and production processes. This means improved collaboration, communication, and coordination between owners, suppliers, users, and operators. The following solutions and actions are aimed at promoting this:

<table>
<thead>
<tr>
<th>Recommended Solutions</th>
<th>Actions</th>
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</table>
| **Integration of activities along the supply chain** | - Introduce the key elements of effective supply chain management (SCM):  
  - a client project manager – critical  
  - a comprehensive project plan, which may be a separate consultant engagement  
  - an effective project brief finalised in collaboration between client and consultant  
  - support for integration along the supply chain by the timely use of techniques such as value management (as opposed to cost management), value engineering, risk management, etc, ensuring participation jointly of key project stakeholders, *i.e.* client end-user, designer, sub-consultants, contractors, operator  
  - engagement of the construction contractor early enough to influence the constructability, *i.e.* at the interface of the design and construction process  
  - a project delivery plan that remains current through the whole of the supply chain management as a living document – make the distinction between documentation for information purposes and for contractual purposes. |
| **Relationship contracts** | - Promote and/or develop a simple methodology for relationship contracting on projects including those under $10 million and negotiate with Government for its testing and implementation. |
| **Facility management and lifecycle management** | - Explore the opportunities for a wider engagement of the industry with clients and owners in facility management and life-cycle management. |
5.3.4 Design issues: Client’s involvement in coordination of project design; inadequate design process

**Root causes:** Lack of a qualified, client-appointed design manager/coordinator to formulate and oversee project integrity and continuity, and inadequate funding for preparation of briefs, programming and management and accountability at critical stages of design, leading to a poor understanding of what is required to optimise designs and provide quality documentation.

Project clients must accept the benefits of staying involved in the management of the project from start to finish, and monitoring the design process through the appointment of a client design manager/coordinator.

Improvements are also suggested for the design process. It is essential that some checking procedure is adopted for every project. The checking process – in effect a second opinion – has traditionally taken two forms. John Carpenter (The Structural Engineer, 3 August 2004) says:

> “The first is a mathematical review to ensure compliance with the relevant Code of Practice; the second is an overview of the major assumptions, of special features, of reserve redundancy, and the other major influences, to minimise the chance of a significant problem. The first check may be performed at peer level; the second can only be performed by a designer experienced in that field. Prudent organisations implement both checks.”

Another simple, economical and effective method of checking documentation before it leaves the office is to use a focus group of three or four experienced personnel working together in a three or four hour session around the conference table. The cost of doing this is insignificant compared to the cost of errors in the document.

The following suggested solutions and actions are aimed at addressing these issues:

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<tr>
<th><strong>Recommended Solutions</strong></th>
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</table>
| Client design manager/coordinator | • Promote the benefits of effective design management to achieve coordination across all parties involved in the construction effort. The role is to maintain the focus on overall project objectives, to identify and remove barriers and to keep the client’s perspective continually in view.  
  • Promote the allocation of a single point of responsibility to ensure that documentation is properly coordinated. |
| Industry recognised design and documentation processes | • Promote among professional bodies across all disciplines  
  - architecture, engineering, quantity surveying and project management – the need to establish industry recognised design and documentation processes and systems including benchmarking techniques to ensure continuous improvement.  
  • Promote discussion within industry associations of past achievements and shortcomings, learning from feedback and changing requirements within the construction industry. |
5.3.5 Rebuilding our human resources

*Root cause: Skill shortages*

The quality of project documentation is being adversely affected by skill shortages in two ways:

1. A general shortage of staff in the industry – a situation which will become worse as the current infrastructure boom extends over the next one or two decades. For example, growth rates in the Australian engineering population will fall to 1.5% a year by the end of the decade, from 4 to 5% a year historically. The number of engineering graduates – about 5000 students per year of whom 1000 are international students who will return home after graduation – will be the same at the end of this decade as it was at the beginning. In Australia in 2004, there were 1825 applicants who met the minimum tertiary entrance requirements and who wanted to but could not get an engineering place at university. In recent years at least four universities have closed engineering courses.

2. A lack of interest in teaching appropriate documentation skills both in on-the-job training and in academic courses.

Short-term solutions need to consider staff who are already trained:

i) a mooted rise of 20,000 skilled immigrants per year (approval of a skilled migration visa takes 4 to 5 months, and there is an upper age limit of 45 years)

ii) greater use of four-year long-stay business entry visas (approval period 10 to 30 days)

iii) a recent relaxation in overseas students studying in Australia permitted to apply directly for skilled migration without having to leave the country

iv) flexibility in working hours and assignments to facilitate (re-)hiring working mothers and mature age workers (45 to 64 years plus) experienced in documentation

v) moves to attract back to Australia experienced practitioners currently working overseas, and allowing overseas based guest workers on major projects.

Medium-and long-term solutions need to focus on in-service and academic training courses similar to those summarised recently by the ACEA (Teresa Charles, Chief Executive ACEA, as quoted in the Australian Financial Review, 4 April 2005). Australia ranked 22 out of 30 OECD countries in the percentage of new science, engineering and building profession degrees in the year 2000. To improve this situation the industry should consider the following initiatives:

i) the Federal Government’s skills audit is a step in the right direction but needs to take account of specific design and documentation skills

ii) there should be priority allocation for Commonwealth supported places across professional design and construction disciplines and support for specialist postgraduate training

iii) incentives should be given to universities with a good track record in graduating women in engineering building professions (in 1997, 20.5% of all students starting engineering classes were women, compared with only 14.2% in 2004)

iv) the current mildly negative perception of engineering building professions among parents and students needs to be modified to encourage more students into engineering as a career building and construction careers

v) more flexible and industry friendly programs should be introduced, e.g. company based training and development, sandwich courses, mature age entry, accelerated learning and bridging programs

vi) more encouragement needs to be offered to the 135,300 trainee apprentices who cancelled or withdrew in the year to 30 September 2004 (this compares to 135,700 who completed their courses)

vii) education to reverse parental attitudes discouraging children from entering the trades.

The following solutions and actions are aimed at promoting the points raised above:
<table>
<thead>
<tr>
<th>Recommended Solutions</th>
<th>Actions</th>
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</table>
| **Competency standards**           | • Encourage the individual professional bodies to review the situation across the industry as to whether significantly increasing the minimum competency standards is required for professional licences, and whether to impose more substantial requirements for Continuing Professional Development (CPD) for continuing registration.  
• Encourage individual professional bodies to develop competency standards for their profession, by a date to be agreed, and to register their standard with the appropriate body. (Several industry bodies already have competency standards.) |
| **Qualified staff**                | • Promote to client bodies the necessity for engagement of adequately qualified professionals and technical staff and the need to regularly assess performance. |
| **Shortage of skilled personnel**  | • Address the shortage of skilled personnel throughout the building and construction industry including such initiatives as industry exchange programs for students and graduates between disciplines, industry-based and in-house training schemes, and strengthening the application of mentoring techniques throughout the industry.  
• Encourage consulting firms to increase the number of cadetships and traineeships. |
| **School programs**                | • Support the various initiatives listed in the introduction to this section.  
• The construction industry as a whole to support this initiative by supporting moves to increase interest among school and college students with such programs as “EngQuest” and “The Science and Engineering Challenge” currently being promoted by Engineers Australia, and the “Futurenet” program promoted by ACEA. |
| **Skill levels**                   | • Ensure recognition for design organisations with high calibre CPD schemes and performance, e.g. a few firms are currently exploring with education institutions if their in-house programs can be developed to provide credit towards university qualifications such as an MBA or a Master of Engineering.  
• Ensure university courses cover ethics and professional standards adequately.  
• Introduce mandatory ethics and professionals standards training (preferably with regular updates) – no registration without completing training. |
### 5.3.6 Making effective use of technology

**Root cause: Inadequate/ineffective use of CAD (Computer Aided Drafting) and ICT (Information and Communication Technology) for design purposes**

Ultimately designs need to be developed by competent designers using design skills and traditional design processes that:

- Use technology as a design tool and not a design process.
- Are based on appropriate procedure checklists.
- Include comprehensive checking of work.

Documentation also needs to be produced by competent practitioners:

- Who are well trained in the use of current technology.
- Who are provided with all the project information available.
- Who are recognised as an important part of the design team.
- Who use relevant checklists and procedures.
- Whose work is independently checked.

The solutions and actions listed below are aimed at ensuring the above:

<table>
<thead>
<tr>
<th>Recommended Solutions</th>
<th>Actions</th>
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</table>
| **Rapidly changing technology** | • Encourage use of compatible software programs that are capable of integrating with each other, and capable of integrating across the different disciplines allowing fast and effective communication.  
• Introduce crosschecking mechanisms and the teaching of rule of thumb checks to CAD personnel who operate CAD systems on the basis of IT experience rather than experience in the relevant professional discipline. |

<table>
<thead>
<tr>
<th>Skill levels (cont)</th>
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</table>
| • Encourage in service training by industry associations and mentoring within the industry.  
• Sponsor formal risk management training to meet the requirements of consultants and the building industry.  
• Create training programs to encourage a co-operative approach to integrating the project phases and to problem solving.  
• Ensure training of new staff provides adequate graduate competency in regard to CAD packages and other technology; and produces competent design and documentation professionals capable of correctly using technology, e.g. by encouraging classes in specification writing.  
• Sponsor formal design management training by industry bodies for consultants and the building industry and up-skilling of design professionals to recognise value added solutions and good documentation.  
• Ensure industry-wide training includes techniques for effective communication. |

<table>
<thead>
<tr>
<th><strong>Actions (cont)</strong></th>
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</thead>
</table>
| • Encourage in service training by industry associations and mentoring within the industry.  
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• Create training programs to encourage a co-operative approach to integrating the project phases and to problem solving.  
• Ensure training of new staff provides adequate graduate competency in regard to CAD packages and other technology; and produces competent design and documentation professionals capable of correctly using technology, e.g. by encouraging classes in specification writing.  
• Sponsor formal design management training by industry bodies for consultants and the building industry and up-skilling of design professionals to recognise value added solutions and good documentation.  
• Ensure industry-wide training includes techniques for effective communication. |
## Recommended Solutions (cont)

**Enhance software for the best design and documentation practice**

- Guide the development of software to meet best design and documentation practice that, for example:
  - uses technology as a design tool not a design process
  - includes appropriate checklists
  - includes comprehensive checking of work, and
  - keeps track of version control including marked up changes from one version to the next
  - enables early involvement of contractors’ and suppliers’ expertise
  - allows integration of data across disciplines
  - enables electronic modelling to:
    - visualise the project for public consultation
    - allow a full appreciation by the constructor
    - check design interfaces in 3-D to avoid clashes
    - produce a reliable bill of quantities
    - validate the design and produce engineering drawings
    - provide asset management data.
  - enable CAD integration with specification text preparation

## Actions (cont)

### Project specific specifications

Project specifications need to be made specific to the project in hand (avoiding the use of “standard” or “off-the-shelf” specifications except possibly in some engineering disciplines).

1. Technological assistance presently comes in the form of Master Specifications which can be electronically downloaded to users’ computers. A number of these word processing Master Specifications are available e.g. from:
   - Natspec (Construction Information Systems Australia Pty Ltd)(see Note 1 next page)
   - SpecPack
   - Aus-Spec

2. A significant technological advancement to word processing based systems are software programs which assemble project specific specifications from databases. These programs are currently in use.

- Encourage and assist Master Specification Developers to:
  - continue to prepare documents that cover all construction disciplines, address present day materials and all construction techniques
  - prepare educational material/tools which assist Specifiers in the competent use of the documents
  - promote the development and use of software programs that have System Authors determining the required text from Specifier’s requirements
  - promote the development and use of CAD / Specification Writing software that enables the integration of drawings and text preparation and subsequent joint access by the Contractor


### 5.3.7 Communicating across the industry

**Root cause: Lack of appreciation of the essential nature of open communication**

Do we need to rationalise the many types of communications used on projects and clarify the legal/contractual certainty of each of them?

- Verbal / phone
- Verbal / face to face
- Letter delivered by post or courier
- Site Instruction, Request for Information, Site Memo
- E-mail / voice
- E-mail / text
- Facsimile
- Web / written
- Web / voice

Within this plethora of communication techniques, it is critical to realise the power and importance of conversations you have or choose not to have. Roger Olds (Engineers Australia, March 2005) identifies five types of conversation:

1. conversation for completion – to ensure the path is clear to move forward
2. conversation for relatedness – to gain alignment and commitment
3. conversation for possibility – to explore what may be done
4. conversation for opportunity – to explore how things may be done
5. conversation for action – to define who and when things will be done and realise that one of the most important parts of conversation is listening.
The proposals below are aimed at restoring recognition that face-to-face communication is essential for sharing of views, debating options, achieving common understanding and assisting with relationship building. They require that the inordinate time/cost pressures commonly experienced are addressed.

### Recommended Solutions

A project communication plan forms part of the project delivery plan and should:

- Set up protocols and conventions for electronic and interpersonal communications.
- Clarify industry/project roles and level of authority and responsibility. A culture of closed communication thwarts early recognition of project issues/problems – these then only surface when a crisis develops.

<table>
<thead>
<tr>
<th>Portal-based communication systems</th>
<th>Pre-start meetings, production meetings and post-completion meetings</th>
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<tbody>
<tr>
<td>Explore the use of portal-based communication systems to improve communications between contractor, site and consultants.</td>
<td>Promulgate communication success stories to support awareness and training activities:</td>
</tr>
<tr>
<td>Write a specification of industry needs for collaboration with CRC and/or software developers.</td>
<td>- “production meetings” to be reinstated with a focus on in-house reviews that ensure achievement of project outcomes.</td>
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<tr>
<td></td>
<td>- establish opportunities for “sharing experiences” with improved communication strategies and techniques.</td>
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<td></td>
<td>- modern innovative communication to be utilised in a manner that augments the fundamental interpersonal processes for communication.</td>
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<tr>
<td></td>
<td>Improve communication in project documents so they are intelligible to all parties, and establish processes that verify the content means the same thing to all parties.</td>
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</table>
5.4 Implementation strategy

5.4.1 Vision for the construction industry

The Task Force’s vision for the future is for an efficient, productive, sustainable and profitable construction sector that will attract the highest quality entrants, through significant improvements in the quality of design and project management documentation, as expressed at the beginning of this Section 5.

A more detailed vision for each of the 10 Core Issues identified by the Task Force is recorded in Section 4:

- For project briefs.
- For integration along the supply chain and between project phases.
- For professional ethics and standards.
- For effective bidding and selection strategies.
- For risk management.
- For the client’s design management.
- For the design process.
- For human resources.
- For managing technology.
- For open communication.

Fundamental change in each of these areas will deliver tangible benefit to the community, clients and industry stakeholders through more stable, reliable productive, efficient and effective procurement of construction.

Issues for action to make this vision a reality, identified by the industry and described in this report by the Task Force are:

- Building stakeholder support and participation.
- Changing industry culture and behaviours.
- Creating a continuous improvement cycle.

5.4.2 Building stakeholder support and participation

Industry stakeholders consist of consumers and producers of construction, with support services.

Consumers of construction, including building owners, developers, financiers, insurers, property professionals, end users and others, influence and are influenced by the construction sector, but they often do not see themselves as “inside the industry”, because construction is not their core business. Their contribution is at the interface with the industry. Their interest is in the delivery of quality, sustainable infrastructure, reliably, effectively and efficiently.

Producers of construction including designers (architects and engineers), managers (project managers, quantity surveyors), contractors (head contractors, subcontractors, trade contractors) and suppliers have a knowledge and understanding of construction processes. Their responsibility is to manage the delivery of construction and the integrity and sustainability of the industry.

Support services include legislators, educators, researchers, suppliers and insurers.

The primary focus of each of the above groups is:

<table>
<thead>
<tr>
<th>Stakeholder Group</th>
<th>Primary Focus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Consumers of construction</td>
<td>• Engaging consultants and contractors</td>
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<td></td>
<td>• Project briefing</td>
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<td></td>
<td>• Design management</td>
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<tr>
<td>Producers of construction</td>
<td>• Sustainable revenue streams</td>
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<td></td>
<td>• Ethics and standards</td>
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<td>• Training and professional development</td>
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<td>• Employment</td>
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<td>• Research and development</td>
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<td>• Technology uptake and diffusion</td>
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<td>• Sustainability</td>
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<td>• Continuous improvement</td>
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<td>Support services</td>
<td>• Education and competence</td>
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<td></td>
<td>• Research and development</td>
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<tr>
<td></td>
<td>• Legislation and regulation</td>
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<tr>
<td></td>
<td>• Technology development</td>
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5.4.3 Managing the task

The task ahead involves confirming existing standards, modifying where necessary, and/or setting new standards, lobbying industry members and government, and monitoring performance. This is likely to be done with a combination of voluntary cooperation and regulation.

Because it is an ongoing task it will require an ongoing organisation. Such an organisation, to be effective, will need the active support of industry leaders and governments. This more formal body may be set up and sponsored by the industry under a Heads of Agreement, or a Memorandum of Understanding.

In the interim, the 18 existing Task Force organisations must commit leadership, resources and sufficient funding to initiate meaningful initiatives.

The immediate role of the organisation is to:

- Establish a detailed Project Plan or Action Agenda, with appropriate strategies.
- Allocate resources to and monitor the progress of the Project Plan or Action Agenda.

The proposed Project Plan / Action Agenda will be set within a four to five year timeframe.

In the longer term, a plan to implement Quality of Documentation nationally must be developed. This is an important step because the waste and other poor consequences of the current situation in Queensland are proportionately more significant nationally. This national plan could include:

- Seeking the cooperation of sister organisations in the other states and parent organisations nationally of the existing stakeholder bodies.
- Collaborating with any parallel efforts in other states.
- Applying for a Federal Department of Industry, Training and Resources Action Agenda for construction documentation.
- Developing or joining a suitable national umbrella body to manage and monitor a national initiative.
- Developing State based bodies, based on a national model, to implement national quality of documentation policy.

5.4.4 Communicating and marketing the message

Many in the building and construction industry – whether client/developer/financier, professional consultant or constructor – are not fully aware of:

- The negative impact of current practices and behaviours on the project delivery process.
- The enormous cost penalties on the industry as a whole, on project owners and on the profitability, indeed the viability of most other parties to the project.
- The angst generated by the present emphasis on adversarial fixed price contracts even in circumstances where a relationship contract would be more appropriate.
- The flow-on effects to other root causes of poor project documentation.

It is imperative that both the impact of poor quality documentation on the industry and external stakeholders, and the benefit of initiatives proposed and developed by the Task Force are communicated clearly and regularly to industry and external stakeholders.

It is proposed that the Task Force develops, as a Project Plan priority, a communications and marketing strategy, to include:

- Public relations strategies.
- Industry information strategies.

5.4.5 Changing industry culture and behaviours

The Task Force will develop, manage and monitor a detailed Project Plan or Action Agenda to drive cultural change and behaviours, based on Section 5 of this report, Recommendations for Industry Change.

The Task Force will identify and utilise existing legislation and regulatory authorities to reinforce, define and mandate appropriate documentation standards. Existing legislation and regulations are listed in this report in Section 5.2.

Priorities will be allocated on the basis of high impact tasks, being those that:

- Significantly improve the quality of project documentation.
- Reduce waste and inefficiency in the industry.

From Section 4 of this report, the priority areas identified by the Task Force are:

- 4.1 – Project briefs
- 4.4 – Selection strategy and bidding philosophy
- 4.3 – Professional ethics and standards
- 4.5 – Risk management
- 4.6 – Design management
- 4.9 – Technology
- 4.8 – Human resource capacity

Issues requiring open and frank debate include:

- The strength of and compliance with codes of ethics and professional practice.
- Undercutting and price competition.
- Legislative, industry and professional sanctions.
- The effect of low-price focus by consumer and competition authorities.
- Complacency and apathy in the industry.
• Management skills in the industry.
• Project procurement skills in client organisations.

5.4.6 Continuous improvement

A continuous improvement cycle will be established by the Task Force or its successors.

The Task Force and its successors will be responsible for:
• Establishing best practice and continuous improvement standards and monitoring processes.
• Maintaining registers of current research into documentation and supply chain processes and practices.
• Maintaining registers of current technologies that support documentation and supply chain processes and practices.
• Maintaining and updating standards and codes of practice in response to feedback cycles.
• Maintaining and monitoring a complaints / compliance register. Liaising with professional bodies in relation to sanctions and corrective action.
• Monitoring accreditation of educational institutions and professional development programs.
• Providing training and professional development in skills shortage areas.

5.4.7 Everyone to play a part

In addition to the responsibility held by the Task Force or its successors to take this work forward, the implementation of change will depend on everyone in the industry. Individuals and organisations can take steps immediately to begin the process of change.

We can all change our approach to project design documentation and become involved in the efforts of our industry association to make the industry-wide and larger changes required to make this better, safer, value for money approach the norm in the industry.
Appendix:
Support from the building and construction industry

Support for the proposition that the declining standard of project design documentation has led to significant waste in the construction budget has been overwhelming. The industry is keen to see some resolution of the problem.

Construction Forum – May 2004
One hundred and twenty senior personnel from all sectors and all disciplines of the industry, working in groups of 10 in an interactive forum, gave close to unanimous support for the definition of the problem, the list of critical issues, and the list of root causes contained in this report.

Consultation with chief executives
Strong personal support was received in face-to-face interviews with the leaders of all of the stakeholder organisations listed on the back cover of the report. The Task Force and its sub groups were drawn from all of these organisations.

Letters of support signed by their state chief executives have been received from the Queensland chapters of:

- The Association of Consulting Engineers
- The Australian Steel Institute
- The Australian Institute of Project Management
- The Australian Institute of Quantity Surveyors
- Civil Contractors Federation
- Queensland Main Roads
- Department of Public Works
- Master Builders Association Queensland
- Queensland Law Society (Construction Council)
- The Royal Australian Institute of Architects

Construction Forum – June 2005
One hundred and fifty senior practitioners from all sectors and all disciplines of the industry, working in groups of 10 in an interactive forum gave strong support, table by table, for:

- The 10 root causes listed in the “Getting It Right The First Time” draft report.
- A proposition that all sectors of the industry have contributed to the problem, and therefore must contribute to the solutions.
- More time being spent on the design phase, including time to review documents properly.
- The vision statements listed against each just cause
- A statement that project briefs are often inadequate, and require professional input.
- Wide use of qualifications based assessment (QBS) of consultant services, but with consideration for new entrants, and with legislated regulation restricted to government works.
- A proposition that’s unethical/unprofessional to tender prices that do not cover the services in full unless the ‘reduced services for reduced fees’ are clearly stated.
- The proposals to address skill shortages, but with a balance between professional, technical and trade skills.
- The creation within the industry of an environment that fosters a passion for excellence – rather than an overly prescriptive externally driven code of practice and/or Australian standard.
- A whole of industry Task Force to promote implementation of the report’s recommendations.
- Effort will be required to overcome the * to changing industry culture such as industry inertia, awareness at all levels, a historic push for lowest price, the will to persist, and the aggressive nature of the industry.

Individual feedback forms
Individual feedback forms were distributed with the draft report of “Getting It Right The First Time” and 31 were returned separately to the feedback at the June 2005 forum. The results, shown on the next page, revealed strong support for the position taken by the draft report. In response to the feedback, the Task Force made a number of refinements to the report.
Feedback Results

1. Do you agree that quality of documentation is a major problem for the construction industry today?

2. Do you accept industry research on the scale of the problem indicating a 10-15% waste of construction resources, equivalent to an annual loss for Queensland of around $2 billion?

3. Do you think the 10 "Root Causes" identified in this report are the critical drivers behind the problem?

4. Do you believe that the problem impacts equally on all segments of the industry – financiers, owners, designers, constructors, suppliers and users – and that all sectors should cooperate in solving the problem?

5. Has the report adequately defined a vision for the future that will result in industry benchmarks for project documentation?

6. Do you fully support an industry-wide effort to improve the standard of project documentation?

7. Do you agree that project briefs are often insufficient to clearly state client requirements?

8. Do you support the use of Qualification Based Selection (QBS) of professional services in lieu of the lowest bid?
9. Should we as industry (consultants, designers, advisers, contractors, and suppliers) be advising clients/owners that we are not prepared to work for them unless they can demonstrate commitment to allocating enough time, resources and funds to ensuring a suitable level and competency of documentation is available at time of committing to the construction contract and its price.

10. Do you support improvement in the following aspects of project delivery:

a) strengthening and/or stronger adherence to existing professional codes of ethics?

b) more effective and equitable risk management?

c) universal use of a client-appointed design manager?

d) the proposals to address the current / looming skills shortages in the industry?

e) improving communication across the industry?

11. Would you support setting up a more formal whole-of-industry task force to promote implementation of the Report’s recommendations?

12. If yes, for how many years?

Answers ranged up to 10 years with an average of 4 years.

13. Would you support preparation of an Australian Standard for project documentation?
Would you support lobbying for legislative action to mandate QBS for public works in Queensland along the lines of the US Brooks Act (1972)?

Are you in favour of:

a) negotiating with the Insurance Council of Australia to reward the highest quality of documentation by a firm with easier availability of / lower premiums for PI cover?

b) voluntary participation in an independently run “AAA” type rating scheme for professional consultants?

c) recommended fee guidelines for professional services?

d) alternatively, strong guidelines for the content / extent / quality of documentation acceptable to industry?

Do you believe the report has identified a credible process for implementation of its recommendations?
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- Queensland Major Contractors Association
- Civil Contractors Federation
- Master Builders Association Queensland
- Property Council of Australia
- Australian Institute of Building

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- Australian Steel Institute, and Queensland Institute of Steel Detailers
- Air Conditioning and Mechanical Contractors Association
- Institute of Public Works Engineers Australia
- Association of Consulting Engineers Australia
- Royal Australian Institute of Architects
- Australian Institute of Quantity Surveyors
- Building Designers Association
- Australian Institute of Project Management
- Queensland Law Society (Construction Committee)

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- Department of Main Roads
- Project Services Division of the Department of Public Works
- Brisbane City Council

The Task Force is grateful for this support and believes the “whole of industry approach” adds a great deal of strength to the recommendations in this report.
Appendix 10

ASI Discussion Paper: 'Are You Getting The Bolts You Specified?'

December 2005
Are You Getting The Bolts You Specified?
A Discussion Paper
STEEL CONSTRUCTION - EDITORIAL

Editor: Scott Munter, National Manager – Engineering & Construction

Over the past two years high strength structural bolts have once again resurfaced as a quality concern due to the market pressure to reduce project costs. This has resulted from the importation of product from new global sources. Bolts are key structural elements and to accept untraceable product without quality certification from a recognised authority is very high risk.

A market perception is that there has been no major failures or life lost as a result of high strength bolt failures. But there has been failures both at installation and in-service many of which are never reported but concealed by the legal system. ASI last reported on failures in August 2001. This litigation process conceals not only the problem but the important lessons and learning. Safety factors are included in design to prevent failure but designers have to remember that inbuilt code and material standard safety factors are for product complying with Australian Standards. These safety factors are reduced even further by nonconforming product.

ASI has held technical evenings in most states and many government authorities have released hazard alerts, planning circulars and building notes to provide advice on high strength bolt quality to all facets of the supply chain, installers and certifiers. It should be stated that there are very reputable quality bolt suppliers still available in the Australian market. This comprehensive paper brings together the how and why this problem has occurred, but more importantly examines the technical aspects of high strength structural bolts. This paper enables readers to understand all critical mechanical properties and geometrical aspects before concluding with guidelines of the minimum requirements for certification. ASI has also included a reference to current media releases at the conclusion of this paper. Members are encouraged to read these additional documents and forward any feedback on bolt related issues in the Australian construction market.

ASI refers readers to the reprinted Advisory Note in this issue on the design methodology for eccentrically connected cleats which now contains recommendations.

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The views expressed in these papers are those of the authors and do not necessarily reflect the views of ASI. Submissions should be in electronic format including all diagrams and equations in two columns, using Times font (size 10.5 min to 12.0 max points). A clean, camera ready printout at 600dpi should also be forwarded.

ASI CONTACT DETAILS

Head Office
Level 13, 99 Mount Street
North Sydney NSW 2060
PO Box 6366
North Sydney NSW 2059
Telephone: (02) 9929 6666
Facsimile: (02) 9955 5406
Email: enquiries@steel.org.au
Website: www.steel.org.au

Queensland & NT
Mr John Gardner – State Manager
Brisbane Technology Park
Eight Miles Plains, QLD 4113
Telephone: (07) 3853 5320
Mobile: 0418 788 870
Facsimile: (07) 3853 5321
Email: john@steel.org.au

Western Australia
Mr John Yeudall – State Manager
Level 3, 220 St Georges Terrace
Perth WA 6000
Telephone: (08) 9480 1166
Mobile: 0419 045 698
Facsimile: (08) 9226 2355

Victoria & Tasmania
Mr Ian Cairns – State Manager
68-72 York Street
South Melbourne VIC 3205
Telephone: (03) 9694 4499
Mobile: 0417 426 002
Facsimile: (03) 9694 4498
Email: ianc@steel.org.au

South Australia
Mr Ken Remphrey – State Representative
3 Harman Avenue
West Beach, SA 5024
Telephone: (08) 8235 0166
Mobile: 0408 833 241
Facsimile: (08) 8235 0172
Email: kerr@steel.org.au

New South Wales & ACT
Mr Brendan Colgrave – State Manager
Level 13, 99 Mount Street
North Sydney NSW 2060
Telephone: (02) 9929 6666
Mobile: 0424 225 701
Facsimile: (02) 9955 5406
Email: brendanc@steel.org.au

STEEL CONSTRUCTION VOLUME 39 NUMBER 2 - DECEMBER 2005
Are you getting the bolts you specified?
A discussion paper
Saman Fernando & Steven Hitchen - Ajax Engineered Fasteners

Summary
Bolted connections have the potential to be the weakest link in structural steel framework if the project does not receive the bolts that are specified in the design documentation. Suppliers must source and certify compliance of bolts with the design specifications. The specification involves a duty of care by the supplier and specifier and if this process does not occur the traceability of the system is compromised and the risk of poor quality product increases as observed in some recent Australian projects. Market pressure to reduce the cost of steel packages has resulted in global sourcing from regions with varying levels of capability and lesser experience in the manufacturing of quality assured structural bolt assemblies. Slightly more expensive product which is a negligible percentage of the overall project cost is cheap insurance to pay for a safe and serviceable structure. Supply chain quality assurance is critical to minimise this quality risk and ensure conforming bolt assemblies are supplied, installed and certified. This paper concentrates on high strength bolt assemblies commencing with a brief outline of the market history of structural bolts in Australia and details quality problems, marking and properties before concluding with an outline of the documentation required to ensure compliance.

Introduction
Bolted joints are designed to keep structures and other engineering assemblies together. Engineers are going to extreme lengths to make sure that the correct fastener is used in the design. In general, the design will perform to the level that the engineer expected only when the bolts specified by the engineer are used as intended. In engineering designs, bolts are specified in various ways. It is common that a particular Australian Standard or a set of standards are referred to when correctly specifying a bolt and a nut. Simple parameters such as:

- property class or grade
- nominal diameter
- thread form
- thread pitch
- head geometry
- nut geometry
- overall length
- threaded length
- grip length and
- plating specification

are specified in the standards. When a washer is used in the assembly the internal and external diameters, hardness, thickness and coating of the washer also need to be specified. Similarly, the mechanical properties such as the tensile strength, yield strength and proof strength are also important when selecting bolts. In order to assure a quality construction or fabrication with safety, economy and functionality, bolts that satisfy the expected performance criteria must be used.

Fastener quality is one of the most overlooked – and in some cases disregarded – aspects in construction and manufacturing in Australia. This complacency is a result of 100 years of relying on Australian manufacturers to supply product equal to the highest quality available anywhere in the world. Unfortunately, this is no longer possible as most standard fasteners are now imported; even Ajax was driven out of the market in 2000-01 by cheap imports.

Ajax was the quality benchmark in Australia for many years, forcing importers to operate their own testing and certification systems to compete. Inevitably, the difference between the quality of the Australian-made product and the imported product became less, and the competitive focus turned to price alone. Unable to compete with price as the major selling criterion, Ajax was forced to withdraw from all standard stocked bolt products in 2001.

As the importers began battling each other, in particular, the value and quality of high tensile and structural bolts dropped. With the industry benchmark removed, the need to ensure and certify the quality of fasteners was removed, and importers began to look for cheaper sources.
In 1999, the US government enacted Public Law 101-592 (1990), also known as the Fastener Quality Act (FQA). This legislation forced US importers to take responsibility for the product they were supplying into the US market. The law was introduced after defective and counterfeit fasteners caused the death of nearly 400 US citizens over 15 years. Manufacturers of substandard bolts were faced with either complying with the new law and absorbing the extra costs, or finding new markets with no quality protection. Some of the ex-suppliers to USA may have ended up in Australia.

In 2002, International Standards Organization (ISO) introduced the ISO/FDIS 16426: Fasteners – Quality Assurance system in order to address similar issues occurring on global markets. This provides fastener quality assurance guidelines that are applicable to suppliers, importers, distributors and purchasers.

One early sign of this emerging quality problem in Australia occurred in a large portal frame in Victoria. Chinese structural bolts were fracturing after assembly, which caused a call to replace all bolts from this source. The incident was noted on the (then) AISC website in August 2001.

More recently, a hangar collapsed at Canberra in late 2004, which prompted an investigation by ACT Workcover. The investigation report found that not only were the bolts not up to the standard, but also that certification of any validity was hard to obtain. The report – HA.32 – recommended that end-users insist on test certificates and statements of conformance for the specific batch of bolts.

Ajax Fasteners Innovations (AFI) the R&D arm of Ajax Engineered Fasteners currently offers consulting and risk assessment services as well as failure investigations to the industry. Our NATA-accredited test facilities and wealth of knowledge in the fastener related technologies allow us to help the industry with various fastening-related issues. Typically, we sign confidentiality agreements before we start any consulting or failure analysis work. As a result, AFI is unable to release exact details of the failures we have come across. However, as the number of such failures is alarmingly growing, we have taken this initiative to highlight the current problems occurring in the industry with respect to substandard bolt and nut products. Although some of these failures are associated with incorrect design and incorrect installation, a significant number of failures were a result of using poor quality fasteners. It should be stated that not all current suppliers of imported fasteners supply substandard products. There are very reputable quality suppliers still available in the Australian market.

There is another type of quality issue emerging in the Australian construction engineering industry. It is common nowadays to have large turn-key projects designed in one country (eg. Japan, USA, Germany) and fabricated in another country (eg. China, India, Taiwan) and installed and commissioned in Australia. As some of the overseas designers are not familiar with the relevant Australian standards they tend to use practices well outside the Australian standards. For example, in some countries A490M (PC10.9) bolts are used for typical structural applications. AS4100 does not facilitate the use of PC10.9 bolts in structural applications. In this case, either the relevant standard for the design should be different or a whole lot of additional supporting information may be required to justify the design methods used to comply with AS4100. The issue of responsibility and liability has to be handled carefully here and the design engineers must take responsibility and liability for all the practices outside the Australian standard.

Furthermore, if local riggers are used for installation they may be neither trained for nor have the necessary equipment to carryout the construction to an unfamiliar standard. Tightening methods, tightening torques and so forth must then be prescribed by the design engineers and appropriate training equipment and auditing should be provided. Most likely in turn-key projects, the whole package including structural bolts is imported and as a result the bolts may not comply with the relevant Australian standards. The regulatory authorities such as NOHS and building inspectors as well as insurance companies would face a dilemma when dealing with these projects.

This discussion paper aims at highlighting the typical problems in the market place; how to identify them and how to avoid them. Furthermore, this paper recommends the methods of specifying the correct fastener and the methods of assuring that the correct fastener (as specified) has been used.

1 James A. Speck, Mechanical Fastening, Joining and Assembly, Marcel Dekker, New York, 1997, p.99-103
What are the common quality problems?
The quality issues seen in fasteners can be broadly classified into systematic and unsystematic process characteristics. Systematic process characteristics are built into the system and are present on each and every fastener. For example, the width across flats of a hexagonal drive bolt is one such characteristic. When the forging process is designed, the tools are made to provide a dimension meeting the lower limit of the tolerance. As tools wear off, this dimension gets larger. Once this dimension reaches the upper limit of the tolerance band, the machine should be stopped and tools should be changed.

Typically, the Statistical Process Control (SPC) method is used in production facilities to maintain the capabilities of the processes and to monitor tool conditions. All dimensions and characteristics that are designed into the tooling and processes belong to this category.

Then there are process characteristics that are not systematic. Non-systematic characteristics include:

- cracks in raw material
- occlusions
- inclusions
- welding points in wire
- sporadic chipping in an extrusion die
- chipped forging die
- parts with missing threads
- threads with pitch errors
- non-helical threads and
- excessive coating thickness, among others.

These problems are more intermittent and very difficult to eliminate unless 100% inspection is used. Some of the above defects can be eliminated by improved process procedures such as when the wire is welded, observe and reject the parts including the weld, or hold few parts in a primary holding tray and visually inspect them prior to releasing them to the main output bucket. Visual inspection of this sort will pick up die chipping, thread faults, wire seams and cracks, etc., so that the sorting may be limited to the few parts in the holding tray. Another source of defects in fastener manufacturing is heat treatment. Firstly, heat treatment requires the use of correct chemical composition in the wire. With incorrect chemical composition it will be extremely difficult to achieve the necessary mechanical properties by using standard heat treatment methods. Secondly, the bolts should be heated evenly so that there is no temperature gradient within a batch or in a bolt. This means bolts should be arranged in a single layer on the oven belt (in case of a continuous flow mesh belt furnace) and the belt speed should be adjusted to make sure that all the bolts reach the required band of temperature for heat treatment at the exit of the oven.

Typically, bolts are then dropped into oil tanks for quenching. If a large number of bolts were dropped in at once the temperature of the oil will increase and the cooling rate of the bolt will decrease. As the cooling rate determines the final hardness, variations of hardness among bolts from the same batch may be experienced. The cooling system used to maintain the temperature of the quenching oil should also be efficient and well maintained. Otherwise, the first part of the batch will have a faster cooling rate than the latter part of the batch due to increased quenching oil temperature. In general, heat treatment process must be carried out with utmost care. If the facility is subject to power interruptions (as reported in some parts of China) it will be extremely challenging to maintain the quality of heat treatment as most of the mesh-belt type ovens work at steady state. Batch ovens on the other hand have a better control; however, they are extremely slow and costly for bolt manufacturing.

As already understood all these practices need extreme care, adequate time, stoppages to the machine, modifications to the tooling and set-up as well as a well trained operator. As a result this will add significantly to the cost of the product. Therefore, it must be recognized that quality comes at a cost.

The following is a list of quality issues experienced by AFI testing services:

**Identification:**
1. Mixed product in a box (with different head marking)
2. Head marking not clearly identifiable
3. Head marking does not comply with the relevant Australian standard
4. The box does not contain traceability information
5. The box carries a statement of conformance to national and international standards without any specific reference to the particular standard.
Mechanical Strength:
1. The tensile strength is less than the specified
2. Yield strength is less than the specified
3. Proof strength is less than the specified
4. Hardness (where applicable) is not within the specified range
5. % elongation at failure is less than the specified
6. % reduction in area at failure is less than the specified
7. Strength under wedge loading is below the minimum required
8. Incorrect decarburization depths
9. Failure occurs by head popping off
10. Nuts stripping

Geometry:
1. Incorrect thread geometry
2. Incorrect head geometry
3. Incorrect shank geometry
4. Incorrect bolt geometry
5. Incorrect nut geometry
6. Incorrect washer geometry

Material:
1. Chemical composition not meeting the relevant standard
2. Splits, occlusions and impurities
3. Forging defects
4. Surface defects

Plating:
1. Plating thickness does not meet the relevant standard
2. Uneven plating thickness
3. Binding threads
4. Plating does not cover the full extent of the fastener
5. Corrosion already starting

The following sections describe the extent and impact of those issues.

Mixed product in a box
(with different head marking)
This is one of the most dangerous findings. What this means is that the product in the box has come from more than one manufacturing source and therefore from many different batches. The traceability of the above product is totally lost. Any quality testing done by the end-user does not give any indication to the quality of the products in the box. This is a characteristic of a supplier who totally ignores all quality aspects. This should be reported immediately and the supplier should be avoided for any further purchases.

Head markings not visible, does not meet the standards
Again in order to identify a quality product the markings have to be clearly visible and should meet the AS4291.1 (metric bolts) and AS4291.2 (metric nuts) or AS 2465 (unified) requirements. Not clearly visible head markings may be an indication of the quality of the product.

Box does not contain quality information
In order to be traceable, the relevant standard and the batch number unique to the batch of bolts should be clearly identified on the box. General statements such as the “product contained in this box meets and exceeds all national and international standards” does not mean much. It must identify the relevant standard. Sometimes, the box may not have enough space to identify all relevant standards. In that case the supplier’s literature should provide a list of relevant standards against the part number of the product. Insufficient quality information on the packaging may also be an indication of the quality of the products.

The tensile strength, yield strength and proof strength are less than the specified by the relevant standard
This is very common in substandard components, mainly due to incorrect material or incorrect heat treatment. In some bolts we have found that the surface hardness is extremely high but the core hasn’t reached the necessary core hardness. We have also tested some bolts which failed at around 70% of the expected strength. Many overseas manufacturers do not have heat treatment facilities in-house. They send product to bulk heat treatment facilities. At these facilities, the ovens are typically overloaded and the product in the middle does not go through the full temperature cycle necessary for heat treatment. Hence, only some of the products meet the specification, in some cases, allowing them to pass the batch by selectively testing such product.

% elongation and % reduction of area not meeting the standard
This is a parameter measuring the ductility of the bolts. Due to incorrect heat treatment, bolts can become very brittle. These bolts typically pass the strength criteria but fail in % elongation and % reduction of area criteria.
Strength under wedge loading
This test loads the fastener while placing a wedge under its head. If the fastener is too brittle, the head will snap off under load. This again is a measure of the ductility and the strength of the fastener.

Failure occurs outside the threaded area
Typical bolt design is made in such a way that the bolt failure should always occur in the threaded area or the shank area. If any bolt is failed in the head-shank intersection either a quality problem or an application problem will indeed be the reason. Occlusions, inclusions, material defects, forging defects, not adequate radius in the corner, are the typical root causes for this type of failure.

Nut stripping
As will be discussed later in the text, nuts are not meant to fail in a bolted joint under any circumstances. A stripped nut is a clear indication of lapse in quality if it failed when used with a correct strength bolt. This is again a common occurrence with some substandard imported products.

Surface Defects
Cracks in the wire, occlusions (trapped gas), inclusions (impurities), chipped extrusion dies and forging dies could cause surface defects. Depending on the extent and location of the defect, it could seriously compromise the performance of a fastener. Typically, surface defects cause increased stress concentration leading to crack initiation and subsequent fatigue failures.

Forging Defects
The forging process forms a piece of cylindrical wire into a bolt. The head area of the bolt will undergo severe plastic deformation. It is very important in tool design to make sure that material flow occurs via the shortest route when forming the final part. Incorrect tool design and forging process may cause surface cracks (due to excessive deformation), cold shuts (material flows over to form a hidden cavity) and other local defects that are systematic to the particular tool design or intermittent due to particular condition of the tool (surface roughness, friction coefficient) or the material. All these defects should be avoided by proper tool design and intermittent microscopic examination of cross-sections on the highly deformed zones of the bolt. Typically, the first off products will be dissected and microscopically inspected for cold shuts, surface cracks and any other systematic defects.

Geometry
There are certain geometrical features that are critical to the functionality of the product. Width across flats, width across corners, thread pitch, thread OD and thread ID, etc, are crucial to assure correct engagement with the tightening tool and the corresponding nut. Length, thread length and shank diameter are crucial for the compatibility of the bolt in the joint; the flange diameter, head and nut height, root radius of the thread, the radius in the head-shank intersection and under-head bearing surface diameter are important for the strength performance of the bolt. All of these dimensions are given a tolerance range by the relevant standard. Bolts and nuts must meet these tolerance limits. Incorrect dimensions may also cause nut/bolt binding leading to galling of mating surfaces.

Material
As discussed earlier, chemical composition and uniformity of material, free from splits, occlusions and any impurities is important to allow proper forging and heat treatment of the bolts.

Plating
The main function of plating is to provide corrosion protection and some lubricity. Certain standards (eg. AS1252) require application of wax to the thread in order to assure correct friction conditions and to avoid galling. This requires:

a) the substrate to be properly cleaned prior to application of plating, and
b) an even distribution of the coating thickness over the entire surface area of the bolt.

If the bolts are not cleaned adequately prior to coating, that will cause coat peeling and produce areas without any coating hence compromising corrosion resistance. Electro-plating inherently attracts more plating thickness to the edges of the product. This typically should not cause a problem as the maximum coating thickness of an electro-plated product is in the order 8 -15 µm. Hot-dip galvanized coatings, on the other hand, are sensitive not only to the cleanliness of the bolt but also to the cleanliness of the galvanizing bath. As this is applied by dipping the bolts in a molten zinc bath gravity effects may also cause uneven coating thickness. Drips and dabs also cause uneven coating thickness in poorly galvanized products. Most threaded fasteners should be centrifuged after
galvanizing to alleviate this problem. Failure to do this will also result in uneven coatings. Poor engagement with the nut and the tightening tool are the main practical problems associated with uneven coatings other than compromised corrosion protection. Most of these defects are visible to the naked eye. Many products have been observed already corroded in the box on arrival.

The impact of various defects on the performance of the bolt may vary depending on the defect. While some of the above defects are somewhat innocent, there are some defects that can totally compromise the functionality and safety of the joint.

When describing plating, another important issue is worth noting. It is well reported that high strength fasteners that has hardness above HRC37 are susceptible to Stress Corrosion Cracking and Hydrogen Embrittlement. If such product comes in contact with hydrogen as a part of the manufacturing process unexpected fractures could occur when in use. Common processes such as acid washing and electro-plating bring these products in contact with hydrogen. It is common practice in the fastener industry to apply Hydrogen Embrittlement Removal (HER) process immediately after the high strength parts come in contact with hydrogen. This process requires baking products at 205 °C for a four – five hour period. All products PC10.9 and higher must have HER process done if they come in contact with hydrogen in the manufacturing process. We have noticed that certain high tensile electroplated products coming from overseas may not have undergone this process. To be prudent PC12.9 and higher products must not come in contact with hydrogen in the manufacturing process. That is why PC12.9 bolts are not available in electro-plated finish.

**How do you specify a bolt, nut and a washer?**
The easiest way to specify all the characteristics of a bolt is by referring to an Australian standard which specifies that type of bolt. Then all you need to specify is the nominal length, nominal diameter, pitch, property class and in some instances the surface coating. That means by specifying only a few parameters you can specify any bolt fully and accurately.
Nominal Diameter
For metric series the nominal diameter is denoted with an M followed by the diameter in millimeters. For example, M20 denotes a 20mm nominal diameter bolt or nut.

In BSW or Unified imperial series, the nominal diameter is given as a fraction of an inch. For example, 3\(\frac{3}{4}\)" means a nominal diameter of 0.75 inches.

Thread Profile and Pitch
Some diameters of bolts and nuts are available in fine and coarse threads. For example, an M16 bolt with a thread pitch of 1.5mm can be specified by:
- M16-1.5p
- M16x1.5, or
- M16-fine.

Similarly, a coarse threaded fastener can be identified by
- M16-2.0p,
- M16x2.0, or
- M16-coarse.

ISO Metric coarse thread profiles are described in AS1275 while ISO Metric Fine thread profiles are described in AS1721.

The imperial range is identified by threads per inch (tpi) or UNC (coarse) and UNF (fine). For example, the BSW thread profile can be specified as either ‘3/4BSW 10tpi,’ or ‘3/4BSW’ for the standard 3/4” diameter bolt or nut. The imperial Unified thread profiles use ‘3/4 UNC-10tpi’ or ‘3/4 UNC’ for the coarse thread and ‘3/4 UNF-16tpi’ or ‘3/4 UNF’ for the fine thread. The imperial BSW thread profile is described in AS3501 and Unified National Thread UNC and UNF is described in AS3635.

Nominal Length of the bolt
Traditionally, the nominal length of the bolt for an application is selected by having at least two full threads protruding beyond the nut in the final assembly. The available length range (preferred and non-preferred) is shown in the relevant standard for the bolt. Typically, the length of the bolt (L) is added to the diameter description by adding x L at the end of the identification. For example, M20-2.5px100 for specifying a coarse thread M20 bolt with a pitch of 2.5mm and a length of 100mm or 3/4 UNC x 6 for specifying a 3/4” UNC coarse thread bolt 6 inches long.

Property Class
According to AS4291.1, all high strength bolts can be identified by the property class generally denoted by ‘PC X•Y’ where ‘X’ x 100MPa gives the nominal tensile strength of the material and ‘Y’ gives 10 x the ratio between tensile and yield strength. For example, ‘PC 8.8’ bolt has a nominal tensile strength of 8 x 100MPa = 800MPa and a yield strength of 0.1 x 8 x 800MPa = 640MPa.

Nuts are specified in AS4291.2 in accordance with the strength of the bolt with which they were designed to be used. For example, a PC 8.8 bolt gets a Class 8 nut and a PC 10.9 bolt gets a Class 10 nut. Note that only one figure is given in the nut class. This is because nuts do not have tensile strengths, but proof loads as their capacity. The Class of a nut is not directly related to the strength as it is with a bolt. An M16 Class 8 nut sees a nominal stress of 880MPa under proof load, whereas the corresponding bolt has a tensile strength of only 800MPa. In another comparison, a PC8.8 M16 bolts (AS1110) has a proof load of 91kN and an ultimate tensile load of 125kN based on a tensile stress area of 157mm2. The corresponding class 8 nut based on the same stress area has a proof load of 138.2kN. When this proof load is applied to the nut for 15 seconds via a hardened mandrel and then released, it should be possible to unwind the nut by just using fingers. This makes sure that the nut thread has not undergone any macro level permanent deformation. This is how it assures that the nut never fails in a proper bolted joint.

When a property class is denoted for a metric fastener it automatically assumes that the part complies with AS4291.1 (bolt) or AS4291.2 (nut). All the detail mechanical properties relevant to the particular property class are described in these standards.

<table>
<thead>
<tr>
<th>Surface Coating Type:</th>
<th>Relevant Australian Standard</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hot dip galvanized coatings on threaded fasteners (ISO metric coarse thread series)</td>
<td>AS1214</td>
</tr>
<tr>
<td>Electroplated coatings on threaded components (ISO metric coarse series)</td>
<td>AS1897</td>
</tr>
<tr>
<td>Hot dip galvanized coatings on fasteners (BSW and UNC threads)</td>
<td>AS B193</td>
</tr>
<tr>
<td>Electroplated coatings on threaded components Part 1 - Cadmium on steel, Part 2 - Zinc on steel</td>
<td>AS K132</td>
</tr>
</tbody>
</table>

Surface Coating Type: Relevant Australian Standard

| Hot dip galvanized coatings on threaded fasteners (ISO metric coarse thread series) | AS1214 |
| Electroplated coatings on threaded components (ISO metric coarse series) | AS1897 |
| Hot dip galvanized coatings on fasteners (BSW and UNC threads) | AS B193 |
| Electroplated coatings on threaded components Part 1 - Cadmium on steel, Part 2 - Zinc on steel | AS K132 |
For UNC/UNF bolts the two strength grades available are Grade 5 (similar to PC8.8) and Grade 8 (similar to PC10.9). These grades are specified in SAE J429 for the bolt and SAE J995 for the nut.

Avoid incorrect terminology such as ‘grade 10.9’ or ‘property class 8’.

The structural bolt code, AS1252, refers to all relevant components: bolt, nut and the washer. They are usually supplied in Australia as an assembly, with the washer and nut already on the bolt.

There has been an issue reported with the AS1252 standard that has caused a reasonable confusion in the market place. The AS1252-1983 was superseded by AS1252-1996 the current standard. This was due to an initiative of the Australian Government to align our standards with the ISO standards. Unfortunately, there were two major differences in the new standard that have caused a significant grief in the market place.

Firstly, the 1996 standard specifies an across flat (A/F) dimension of 34mm for M20 bolts compared to 32mm specified in the 1983 standard. When the new standard was released, neither the bolts nor the spanners made to the new standard were available. In Australia 34mm spanner does not come with a standard spanner set and until recently were not available in the market. Currently most countries including USA still uses 32mm A/F dimension on M20 bolts. Hence bolts and spanners made to 34mm A/F dimension was a rarity. As a result the new standard did not come into acceptance for M20 bolts. Progressively, the products made to 1996 standard became available. If available, it is better to use the 1996 M20 bolt as it is more robust compared to the 1983 product. Unless the product is available in the market it is difficult to enforce the new standard.

Secondly, the AS1252-1996 standard reduces the hardness requirement for hot-dip galvanised washers. The 1983 standard specified a hardness range of 35 - 45HRC for all washers. In the 1996 standard, while the same hardness range was kept for non hot-dip galvanised washers the hardness range for hot-dip galvanised washers has been increased to 26 – 45HRC. The specified hardness range for PC8.8 bolts according to AS4291.1 is 23-34 HRC. If the bolts were made to 34HRC they will definitely scour the washers made to 26HRC hence making the washer not as effective. Therefore, it is always better to specify and use the hot-dip galvanised washers made to hardness range 35-45HRC. This may be an additional requirement the engineer may place on the supplier of the products. As the requested range is within the larger range specified in the standard all scrupulous suppliers should have no issues in supplying to the reduced range.

In some cases (where assemblies are not available), when specifying high tensile bolts (to AS1110 or AS1111) the corresponding nuts should be specified to AS1112. The property classes or strength grades must match between the bolt and the nut in order to achieve the full capacity of the bolt. High tensile bolts must never be used with lower property class or grade nuts.

**Nut and Bolt Compatibility**
The correct nut must be specified with the selected bolt. The following table shows typically compatible nuts.

<table>
<thead>
<tr>
<th>Bolt Strength</th>
<th>Matching Nut</th>
</tr>
</thead>
<tbody>
<tr>
<td>AS2451 Mild Steel BSW</td>
<td>AS2451 28tonf proof load</td>
</tr>
<tr>
<td>AS4291.1 Property Class 4.6</td>
<td>AS4291.2 Class 5</td>
</tr>
<tr>
<td>AS4291.1 Property Class 5.6</td>
<td>AS4291.2 Class 5</td>
</tr>
<tr>
<td>AS4291.1 Property Class 8.8</td>
<td>AS4291.2 Class 8</td>
</tr>
<tr>
<td>AS4291.1 Property Class 10.9</td>
<td>AS4291.2 Class 10</td>
</tr>
<tr>
<td>AS4921.1 Property Class 12.9</td>
<td>AS4291.2 Class 12</td>
</tr>
<tr>
<td>SAE J429 Grade 5 UNC/UNF</td>
<td>SAE J995 Grade 5 UNC/UNF</td>
</tr>
<tr>
<td>SAE J429 Grade 8 UNC/UNF</td>
<td>SAE J995 Grade 8 UNC/UNF</td>
</tr>
<tr>
<td>Black Structural Bolts AS1252</td>
<td>Black Structural Nuts AS1252</td>
</tr>
<tr>
<td>Galvanised Structural Bolts AS1252</td>
<td>Galvanised Structural Nuts AS1252</td>
</tr>
</tbody>
</table>

**How do you know you have the correct bolt, nut and the washer?**
Bolts and nuts must contain marking specified in the relevant standard. By inspecting the parts for the correct marking you will have some indication if the product supplied is what you specified.
Unfortunately, the presence of markings alone does not guarantee that all necessary mechanical and dimensional properties conform to the standard. The following tables show the specified markings on bolts and nuts.

### Bolt Markings:

<table>
<thead>
<tr>
<th>Bolt Marking</th>
<th>Bolt Type</th>
<th>Relevant Standard</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hexagon Head Unified National Head Tensile Grade 8</td>
<td>AS2465</td>
</tr>
<tr>
<td></td>
<td>Hexagon Head Unified National Tensile Grade 5</td>
<td>AS2465</td>
</tr>
<tr>
<td></td>
<td>Hexagon Head Precision Metric High Tensile Property Class 8.8</td>
<td>AS1110</td>
</tr>
<tr>
<td></td>
<td>Hexagon Head High Strength Structural PC 8.8 (FJ stands for Friction Joint but not mandatory)</td>
<td>AS1252</td>
</tr>
<tr>
<td></td>
<td>Hexagon Head Metric Commercial Property Class 4.6</td>
<td>AS1111</td>
</tr>
<tr>
<td></td>
<td>Hexagon Head BSW Mild Steel</td>
<td>AS2451</td>
</tr>
<tr>
<td></td>
<td>Inch stud bolts Grade B8</td>
<td>AS2528</td>
</tr>
<tr>
<td></td>
<td>Metric hexagonal coach screws</td>
<td>AS1393</td>
</tr>
<tr>
<td></td>
<td>Tower Bolts</td>
<td>AS1559</td>
</tr>
<tr>
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<td>AS1559</td>
</tr>
</tbody>
</table>

### Nut Markings:

<table>
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<th>Nut Type</th>
<th>Relevant Standard</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hexagon Metric Property Class 8</td>
<td>AS1112</td>
</tr>
</tbody>
</table>

Marking dot may be replaced by manufacturer’s mark ‘X’ here

Hexagon Metric Property Class 4 (Clock-face system)
Ref. small line at 8 O’clock position with respect to the dot representing 12 o’clock position. This system may also be used with bolts.

Hexagon Metric Property Class 5 (Clock-face system)

Hexagon Metric Property Class 6 (Clock-face system)

Hexagon Metric Property Class 9 (Clock-face system)

Hexagon Metric Property Class 10 (Clock-face system)

Hexagon Metric Property Class 12 (Clock-face system)

Hexagon Metric Property Class 5-Style A

Hexagon Metric Property Class 5-Style B

Unified Hexagon (UNC/UNF) strength Grade 5-Style A

Unified Hexagon (UNC/UNF) strength Grade 5-Style B
How do you make sure that the bolts you are purchasing meet the relevant standard?

This is not easy as there is no method of non-destructively testing a bolt to see whether it contains any mechanical or manufacturing defects. In this case, the quality systems of the manufacturer and supplier have to be relied upon. When a supplier claims that a particular bolt, nut or washer satisfies a certain standard (should be same as that specified by the engineer), it is the supplier’s responsibility to provide documentary evidence of conformance and also it is the purchaser’s right to insist on such documentation.

Market experience is that one of the following could happen when asking for a test/compliance certificate from a supplier. They can:

- refuse to provide any documentation
- give you a test report but not a compliance or test certificate
- give you an inspection certificate
- give you a test/compliance certificate from an unaccredited foreign test laboratory

If you don’t know exactly what you are looking for it will become very difficult to accept or reject the documentation provided by the supplier.

The correct test certificate should contain all the information listed below while a compliance certificate should contain at least the information marked bold in full:

- Identification and address of the supplier
- Identification and address of the test laboratory and accreditation seals of the test laboratory
- Date of issue, page number on each page
- Test certificate number
- Batch identification number
- Product identification
- Customer purchase order number to match the batch number and
- Any other system reference numbers.

These make sure that the product is fully traceable from the customer purchase order to the original steel used for the production of the products.

- Test, test specification, measured values in comparison to specification
  Typical for the bolt: tensile test/surface hardness test, raw material specification, reference number and the heat number with chemical analysis or any traceable pointer to this information
  Typical for nut: hardness test, reference number, any other associated test certificate number, raw material specification, reference number and the heat number with chemical analysis or any traceable pointer to these information
  Typical for washer: hardness test, reference number, any other associated test certificate number, raw material specification, reference number and the heat number with chemical analysis or any traceable pointer to these information
- Statement of compliance referring to a definite relevant standard
- Signature of authenticity

Any further information may be requested as agreed with the supplier. Suppliers may charge extra for requested additional information which are not typically provided in the compliance certificates. The fact that a compliance certificate is issued by a NATA accredited laboratory with a signature
of the relevant NATA signatory assures that the certificate is issued as per the quality accreditation guidelines of NATA. A statement of compliance on the certificate assures that the product is in full compliance with the relevant standard. Therefore, lack of certain test results should not cause a major concern. No compliance certificate can be issued if the product does not fully satisfy all the requirements of the relevant standards.

It is very important to identify the difference between a test/compliance certificate and a test report. An independent NATA accredited laboratory may test a few bolts and issue a test report signed by a NATA signatory. A test report may only provide specifications and test results pertaining to the tested samples of the bolt. It may also refer to a relevant standard when quoting the specifications and all the tests conducted and their results may confirm to the relevant standards. This test report cannot replace a compliance/test certificate. The difference is that a compliance/test certificate not only tests the sample bolts but also investigates the process by which the bolts were made to ensure that it is quality-traceable and the sample results can be applied to the whole batch of the bolts.

According to the statistical process theory any number of sample tests (unless 100% of the batch is tested) will not assure the quality of the batch if the batch did not come from a stochastic process where the measured variables follow a known statistical distribution. For example, if the bolts in a box contain three different head markings, that indicates that the bolts did not come from the same process and hence the statistical parameters established using the samples in this box cannot be used to predict the behavior of such processes and therefore the quality of the untested bolts remains unknown.

The above discussion is extremely important in the quality assurance process of bolts, nuts and washers. Sample testing by a local NATA-accredited laboratory may be used as a check for a compliance certificate issued by another non-accredited (in Australia but accredited overseas) laboratory. Even in this process if the local accredited laboratory wants to endorse the compliance certificate issued by the overseas laboratory they must have confidence that the production process is quality traceable and the batch shares the same statistical characteristics of the samples tested.

Without this assurance it is not possible to endorse a compliance certificate. Any issuer of a compliance/test certificate takes responsibility for following the necessary evaluation processes prescribed by the accreditation organisation. When issuing a compliance/test certificate, the NATA signatory accepts liability for any loss or damage caused by issuing a false compliance certificate.

Who is responsible? What can be done?
In any engineering development there are many parties involved with each party having various aims and objectives. For example, a building construction project could involve:

- owners
- architects
- engineers
- quantity surveyors
- builders
- fabricators
- sub contractors
- project managers
- purchasers
- quality auditors
- building inspectors.

Each party brings in a certain expertise to the project and they take responsibility for the work they do. Achieving a well-engineered structure with functionality and economy requires each of these parties working together in the best possible way.

In the case of fasteners, engineers design the joints and specify the fasteners. They expect that the fabricators and builders will use the specified fasteners in the way intended by the engineers. In many projects, however, purchasers buy the fasteners and if they do not understand the engineer’s specification or if the engineer has not specified it accurately; or they are under financial pressure, purchasers may end up buying substandard fasteners. In many cases purchasers are evaluated by how much money they save on the project and it becomes their prerogative to buy the cheapest product especially if they do not understand the required quality. When a failure occurs, engineers or the builders are usually the first to be blamed. What is often evident in this type of conflicts is a lack of clear definition of responsibility. It is paramount that a line of responsibility is properly defined and understood.

USA Experience
When the US implemented the Fastener Quality Act, the mechanism they used was to pass the responsibility to the purchaser.
They identified that regulating a large number of manufacturers and suppliers would be nearly impossible and would also make the low end users (e.g. people using bolts for gardening fixtures, non-critical structures etc.) pay a higher price for quality that they do not need. This also assures the survival of low-quality cheap suppliers.

The fastener quality standards were well defined, publicised and were understood by the engineers, purchasers and testing organisations. Accredited testing organisations were appointed to take responsibility for issuing compliance/test certificates. The legislation enforced civil action against the purchaser in case of loss or damage or simply not meeting the standard. It has been reported that this system works relatively well.

In Australia
Learning from the USA experience, it is our proposal that the fastener quality should be legislated in Australia. Again, the responsibility in the long term must be passed on to the purchaser.

However, in the short term this may not be possible or practical as the purchasers may not have the necessary technical know how to make sure that the fastener quality is correctly assured. In this case, it would be better, in the interim, to pass the responsibility to the design engineers as they are the only party currently having an intimate understanding of the fastener specification, their identification and desired function. The purchaser should have an involvement as they make the final decision to place an order. It is proposed that the purchaser consults the design engineer for certification before placing an order. The received goods should be inspected and signed off by the design engineer.

On a secondary level assurance, the building inspectors may be required to look at the quality documentation relevant to the fasteners. In this case, the inspectors need to be educated to read and inspect the relevant information.

Conclusion
- Fasteners keep structures together. Without fasteners it will become disfunctional
- The fastener quality is of utmost importance to the safety of the community
- Incidence of failures due to poor quality of fasteners is becoming more frequent
- There is no quality benchmark present in the industry
- The current standards provide necessary guidelines to identify the quality of fasteners
- Engineers must be more familiar with providing a tighter specification and purchasers and inspectors must be more proactive in identifying the fasteners and demanding the quality documentation from the suppliers
- Quality assurance cannot be achieved by just testing samples. The manufacturing and distribution processes must be traceable to apply statistical quality control methods
- In a project, there is no particular person responsible for the quality of the fasteners. The responsibility is distributed amongst manufacturers, suppliers, engineers, contractors, fabricators, owners, inspectors and purchasers
- Learning from the USA experience, fastener quality should be legislated
- The ultimate responsibility should be passed to the purchaser in the long term
- In the short term, purchasers should consult the design engineer for certification before placing an order. The received goods should be inspected and signed off by the design engineer. The ultimate responsibility should still lie with the purchaser
- Building inspectors may provide secondary level checking and monitoring
- If adequate measures are not taken as a matter of urgency, it is just a matter of time before a major disaster could occur! Most failures are covered-up on-site and not reported!
**High Strength Structural Bolts**

**Additional references and information:**

*Scott Munter – Australian Steel Institute*

**ASI call for market feedback and information:**

At the time of researching and editing this paper the following information has been sourced in the public domain on high strength structural bolts. Australian Steel Institute encourages members to read these references as they reinforce the duty of care that is required by all sectors of the industry to maintain quality high strength bolt assemblies. ASI encourages any sector of the industry to forward recommendations, reports or incidences regarding high strength structural bolts to Scott Munter, National Manager – Engineering & Construction by email: scottm@steel.org.au This will allow ASI to monitor the performance of the high strength bolt market and report back to industry if any problems persist.

**ASI 2001 website release on imported structural bolt failures**

A failure of imported high strength structural bolts was posted on the ASI website in August 2001 following the dramatic failure at some stage after the erection of a large portal frame in Victoria.

This alert is no longer posted on the ASI website due to the age of this document but is still available on request.

Email: scottm@steel.org.au

**ACT WorkCover – HA.36, Failure of Structural Bolts**

Issued 6 September 2004 this hazard alert followed and ACT WorkCover investigation into the collapse of a steel structure. This collapse identified a number of safety issues relating to metric Property Class 8.8 structural bolts.

This hazard alert is available for download at: http://www.workcover.act.gov.au/pdfs/hazalerts/HA32-BoltFailures.pdf

**AJAX Engineered Fasteners – Technical Bulletin No. 1-05, Nut Compatibility**

Issued 5 May 2005 this report covers the importance of the compatibility of nuts for use with high tensile bolts and provides a case study demonstrating this issue. Both distributors and end-users need to be aware of the need to use the correct nut with heat-treated bolts. This bulletin has been produced in response to complaints about some industry practices.


**Department of Housing and Works – Government of Western Australia. Building Note Number 38-2005, Structural Steel Bolts**

Issued 1 June 2005 by the Government of Western Australia was an alert to the industry on concerns relating to the conformance of structural steel bolts being imported into Australia.

This building note is available for download at: http://www.dhw.wa.gov.au/200506_BN38_Structural_Steel_Bolts.pdf

**NSW Government - Department of Planning. Planning Circular. Structural Steel Bolts**

Issued 19 December 2005 this planning circular was released to provide councils, accreditation bodies, relevant government agencies, industry groups and industry practitioners with advice regarding the quality of structural steel bolts.

This planning circular is available for download at: http://www.planning.nsw.gov.au/planningsystem/pdf/brans/bs05_002_steelbolts.pdf

**HOBSON ENGINEERING**

The HOBSON update being the latest newsletter from Hobson Engineering: Vol 21 (purchasers of high strength structural bolts) has the lead article in this issue from the desk of Peter Hobson titled: ‘AS1252 Structural Assemblies’. It covers the market issues through to a suggested solution.

**ASI Advisory Note**

**Design method for eccentrically connected cleats not to be used**

In Steel Construction Vol. 38 No. 1, March 2004, the Australian Steel Institute published a warning about eccentric hollow section bracing connections. At this time the ASI website also contained the warning. The warning stated that the method for calculating the compression capacity of overlapped gusset plates or “eccentrically connected cleats” may be unconservative. The method is found in *Design of Structural Steel Hollow Section Connections*, the hollow section design manual published in 1996 by the Australian Institute of Steel Construction, now the Australian Steel Institute. The problem arises because of the assumption that “the connection may be treated as two eccentrically connected cleat components whose ends are fixed and prevented from sway”.

(See Note 1.)

The connection types to which this advisory applies are the slotted tube, welded tee end, and flattened end connections. Unless restrained against sidesway, each of these connections deflects laterally as it is loaded in compression, developing a plastic hinge in each plate at a fraction of the section compression capacity. The real capacity of the connection is very much less than would be computed assuming the presence of lateral restraint or the absence of eccentricity. The problem is exacerbated for connections in short compression members and for compression members that are not exclusively wind bracing.

The small eccentricity occurring when a stiff member is connected to a gusset plate (e.g. channel web bolted to gusset plate) has traditionally been ignored in the design of simple bracing connections and this is permissible in some cases because most of the eccentricity moment acts on the stiff bracing member and only a small bending moment acts on the flexible cleat component. There is an important difference between this situation and that with hollow section bracing connections. In these connections there are two flexible components bolted together – “eccentrically connected cleats”. The problem is that the eccentricity moment is shared between the two flexible plates and plastic hinges develop at a very low load unless there is lateral restraint. The cleat assembly deflects sideways during loading. Eccentrically connected cleats should not be designed as a concentric column even when a large effective length factor is used. It is necessary to apply the existing design code rules for combined bending and compression (AS 4100 Section 8 – Members Subject to Combined Actions). Software is available to perform the necessary code checks.

**Recommendations**

- Do not use an eccentric hollow section bracing connection for a short compression member unless it is stiffened against sidesway. A *concentric connection should be used if there is no sidesway stiffening*.
- Design eccentric hollow section bracing connections taking eccentricity into account by rigorous application of design code rules for combined bending and compression – do not use the method in *Design of Structural Steel Hollow Section Connections*.

**Scott Munter**

National Manager – Engineering & Construction

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| 08 9439 1934 |
| **Belliotti & Co**  
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| 08 9414 9422 |
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Canning Vale DC WA 6970 |
| 08 9256 3311 |

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QLD 4008 |
| PO Box 1311 Eglea Farm QLD 4009 |
| www.indgalv.com.au |
| **Orcon Pty Ltd**  
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| PO Box 295 Salisbury QLD 4107 |
| www.orcon.com.au |
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| PO Box 246 Sunnybank QLD 4109 |
| **Fielders Steel Roofing**  
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| www.fielders.com.au |

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| GPO Box 18207 Melbourne VIC 8003 |
| www.bluescope.com |
| **G.A.M. Steel Pty Ltd**  
557 Mount Derrimut Road, Derrimut VIC 3030 |
| PO Box 171 Deer Park VIC 3023 |
| www.gamsteel.com.au |
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| **Smorgon Steel Group Ltd**  
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Port Melbourne VIC 3207 |
| www.smorgonsteel.com.au |
| **Smorgon Steel Reinforcing & Steel Products Division**  
Level 1, 668 Lorimer Street  
Port Melbourne VIC 3207 |

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142-146 Fairbank Road, Clayton South VIC 3169 |
| Private Bag 155 Clayton South VIC 3169 |
| www.webforge.com.au |

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| 08 9356 1566 |
| **JV Engineering (WA) Pty Ltd**  
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| **United KG**  
PO Box 219  
Kwinana WA 6167 |
| 08 9499 0499 |
| **Uniweld Structural Co Pty Ltd**  
10 Malcolm Road  
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| 08 9493 4411 |

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| www.bisalloy.com.au |
| **BlueScope Lysaght**  
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| GPO Box 2695 Sydney NSW 2000 |
| **Coil Steels**  
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| PO Box 166 Granville NSW 2142 |
| www.coilsteels.com.au |
| **Graham Group**  
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| PO Box 57 Yagoona NSW 2199 |
| www.grahamgroup.com |
| **Horan Steel Pty Ltd**  
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| PO Box 8427 Wetherill Park NSW 2164 |
| **OneSteel Limited**  
Level 40, 259 George Street, Sydney NSW 2000 |
| GPO Box 536 Sydney NSW 2011 |
| www.onesteel.com |
| **OneSteel Market Mills**  
Ingall Street, Mayfield NSW 2304 |
| PO Box 245C Newcastle NSW 2300 |
| **Southern Steel Group**  
319 Horsley Road, Milperra NSW 2214 |
| PO Box 342 Panania NSW 2213 |
| www.southernsteel.com |
| **Stramit Building Products P/L**  
Level 5, Tower A, Zenith Centre, 821 Pacific Hwy, Chatswood NSW 2067 |
| Locked Bag 7013 Chatswood DC NSW 2067 |
| www.stramit.com.au |
| **Weldlok Industries Pty Ltd**  
117-153 Rockwood Road, Yagoona NSW 2199 |
| PO Box 57 Yagoona NSW 2199 |
Appendix 11

Defective Chinese Hollow Sections Exported to the USA
Proper Hollow Section Manufacture Shown Above

Typical HSS seam weld face with no visible defects.

Typical HSS seam weld root with no visible defects.
Grind marks indicate apparent repair attempts. Longitudinal cracks, underfill, & incomplete fusion exist.

Defective Imports to USA
Defective Imports to USA

Split / complete lack of fusion.

Incomplete Joint Penetration
Defective Imports to USA
Appendix 12

Extracts from John Scarry Submission on Building Amendment Bill (No 4)
Date: 7 June 2012

(Extracts from) SUBMISSION

To the Local Government and Environment Committee

on the Building Amendment Bill (No 4)

B. General Comments

4. On the first page of the Building Amendment Bill (No 4) (322-1), under ‘General policy statement,’ it states:

‘This Bill:
• introduces enhanced and more comprehensive consumer protection measures, including mandatory written contracts for work valued over a prescribed amount, mandatory disclosure of certain information by building contractors, and new offences for breaches of these requirements:’

(my italics).

As explained below, this is a completely misleading statement. Far from enhancing consumer protection, this Bill, (and its sister No. 3 Bill) adds no effective new protection to a household owner, and actually reduces the means by which an aggrieved owner can seek redress for significant financial loss and hardship.

5. If the construction of a new house or a major modification to an existing house turns into a disaster, whether due to weathertightness defects, structural defects or, most likely, both, how is the owner in any better position than at present? If the contractor won’t or can’t rectify the problems, the owner will have to seek compensation through arbitration, the Weathertight Homes Resolution Service or the courts, exactly the same as at present.

This will take years, and costs many tens of thousands of dollars, if not many hundreds of thousands of dollars, in legal and expert witness fees, before a decision is even made. How is this any different than at present?

What if the contractor is largely liable but no longer exists, or has no money? Where is the mandatory bonding/surety/insurance that stays in place at least 10 years after Practical Completion? There is none. How is this any different than at present?

This whole situation is, to all intents and purposes, the same as it was prior to the No. 3 Bill that recently passed.
Except that the one ‘lifeline’ that has existed for many people in desperate circumstances, a claim against the Building Consent Authority, is being manoeuvred out of reach.

6. It has been stated in writing and in the House that this Bill, and the preceding No. 3 Bill, are essentially ‘building blocks,’ part of a complete ‘structure’ of measures designed to enhance the building and construction industry in New Zealand. As Hon Maurice Williamson pointed out in the House recently, the building and construction industry in NZ is huge, and vitally important to the NZ economy.

To continue the building analogy, these ‘building blocks’ are unfortunately defective materials, and the complete ‘structure’ is built on weak and shifting, liquefiable sand, it is a ‘leaky building,’ a gravity hazard and earthquake prone.

D. Specific Issues With This Bill

20. Contrary to the extremely misleading statement that this Bill “introduces enhanced and more comprehensive consumer protection measures,” this Bill simply maintains current practice, and provides no new protection to the owner should building works go awry.

Having said that, the disconnect between the rhetoric in favour of this Bill, and what I read, or more importantly don’t read, has me worried. Am I missing something?

21. Part 4A – Consumer rights and remedies in relation to residential building work

How, in any meaningful, material way, does this Part change what has existed under common law and related statutes since New Zealand became New Zealand, or, going back to Britain, for hundreds of years?

22. Section 362E

A written contract is essential for virtually all building work, but it plays virtually no part in ensuring proper design and construction. Its only real use with regard to such matters is when the whole thing ends up in court.

Major ‘leaky’ apartment buildings include:

- Farnham Terraces, Parnell - $15 million to fix,
- Nautilus, Orewa – Up to $19 million to fix,
- Sacramento, Botany Downs - $19.2 million claimed,
- Hobson Gardens, Hobson Street – Up to $20 million to fix,
- Pepperwood Mews, Waitakere – a complete demolition job.

All of these projects would have had written contracts.
Refer Sections 17(a) to 17(s) of this submission. All of these shockers had written contracts.

Clearly, written contracts provide absolutely no guarantee of sound design, construction or oversight.

23. **Section 362H – Implied warranties for building work in relation to household units**
   How, in any meaningful, material way, does this change what have been the responsibilities of the various parties concerned, under common law and related statutes since New Zealand became New Zealand, or, going back to Britain, for hundreds of years before that?

24. **Section 362I – Proceedings for breach of warranties may be taken by non-party to contract**
   Presumably, this does not exclude proceedings through the courts. How, in any meaningful, material way, does this section change the status quo in any real and meaningful way?

25. **Sections 362L and 362M – Remedies**
   How is this any different, in any real and meaningful way, to the law relating to building defects that has applied for hundreds of years?

   How does the client obtain reimbursement of costs without taking proceedings described in Section 362I of the Bill, or to the courts?

   Such proceedings invariably take years, and are financially crippling to most owners. How does this Bill change this reality in any real and meaningful way?

   What technically competent, independent, swift acting investigative body, capable of awarding damages in its own right, is set up to ensure these remedies are provided without a delay of years and costs usually amounting to many tens if not hundreds of thousands of dollars.

26. With regards to the WHRS and the courts, how does this suite of legislation even attempt to address the issue of the near complete lack of technical knowledge on the part of the adjudicators or judges.

   With regards to the WHRS and the courts, expert witnesses are supposed to be impartial experts of value to the adjudicator or judge, irrespective of who pays their fee. Unfortunately, my direct experience, and that of other engineers I know, is that usually the ‘experts’ for the defendants in such cases are nothing more than ‘hired guns.’ Much of what these supposed experts state should have them facing disciplinary proceedings or get them struck off. How is this travesty addressed by this legislation?

27. **Mandatory Contractor Insurance**
   Commiserations are all well and good, but the one thing that is essential to fix any building defect or total project disaster is *money*. Unusually, lots of it.
Architects and professional engineers generally carry, in effect are required to carry, at least some Professional Indemnity insurance, although often significantly less than the total project value.

In some jurisdictions overseas, including several US states, contractors are required to be both licensed and bonded, and to work without either is a serious criminal offence. The public there demands it.

Where is the requirement to have mandatory contractor insurance/surety/bonding to cover the full value of the work and potential associated damage for at least 10 years after Practical Completion? Insurance/surety/bonding that survives any form of contracting company liquidation, and is free of all legalistic rorts?

Where is the requirement for damages for real deficiencies to be paid out swiftly upon independent, competent verification of the defects, without the need to spend hundreds of thousands of dollars the owner simply does not have to fight court or WHRS battles?

Without such insurance, and especially as BCA’s are being deliberately manoeuvred clear of being ‘last man standing,’ far from improving protection to home owners, this suite of Bills is reducing protection, whilst dishonestly claiming the opposite.

28. **Section 362P – Building contractor must remedy defect notified within 1 year of completion**
   The heading should have ‘that is’ inserted between ‘defect’ and ‘notified.’

Where is ‘completion’ defined? I could not find it in the Building Act 2004, the Building Amendment Act 2012 or this Bill.

Is it when that part of the work is completed, or ‘Practical Completion,’ or is it when the CCC is issued?

Why is it that defects only have to be rectified when they are notified within 1 year of ‘completion,’ when claims in tort can be taken within 10 years of the act or omission under the Building Act?

What is ‘a reasonable time?’ I can think of many very serious defects in house construction which left uncorrected could cause considerable damage, yet could be corrected within weeks if not days. Defects must be rectified as soon as it is feasible to do them, irrespective of other work commitments by the contractor.

In fact, if a contractor has outstanding defects, he/she should be barred, by law, from carrying out any new work until such time as those defects have been rectified. This would have to be ‘fine tuned’ in any legislation, but you get the idea.

There also exists the contradictory requirement, under Section 362Q (1) (c), that there is an implied onus on the owner to repair any defect as soon as practicable after the defect becomes apparent, for fear of allowing the original contractor to
avoid liability, yet that same contractor can take a *reasonable time* to repair the same defect.

All too often in countries of our type, and especially when the worst building defects are involved, a ‘reasonable time’ for repairs turns out to be a very long time indeed.

29. **Schedule 1 – Building work for which building consent not required** – **Section 14 – Penetrations with maximum diameter of 30 centimetres**

   Although I think the intent is to allow relatively small penetrations through the *exterior envelope* of a building, or *internal partitions* and the like, it could easily be interpreted to allow holes up to 30cm diameter to be cut through structural floor joists, beams, columns and walls. Also, no mention of very common rectangular shaped penetrations is made.

   Such weakening, even endangerment, of the structure has occurred many times, including, in one instance, the cutting of a chase into a reinforced concrete column in a multi-storey building in NZ that almost led to a structural collapse.

   The photos in Appendix G show penetrations that were cut through critical sections of a coupled concrete shear wall, which provided the seismic and wind resistance to a building, without engineering approval.

   This needs to be reworded to clarify that it applies only to penetrations through the non-load bearing ‘fabric’ of a building, and no such penetrations are to affect any of the building structural elements.

   This section should also be expanded to include, say, rectangular penetrations where all sides are less than or equal to 30cm in length.

   Also, ‘centimetres’ are not usually used in the NZ construction industry, as opposed to continental Europe; ‘metres’ and ‘millimetres’ are used in NZ, therefore ‘300mm’ is more appropriate.
E. Some Comments on Building Amendment Act 2012 [Building Amendment Bill (No 3)]

30. I should apologise for not having made a submission on the ‘sister building block’ to this Bill, the Building Amendment Bill (No. 3).

However, given the fact that my efforts for reform over the last 10 years have been deliberately ignored and subverted by those charged with a fiduciary duty to protect the interests of the people of New Zealand, you can appreciate why I did not.

But, since it is ‘one building block’ of a whole structure, and it is inextricably linked with this Bill, I shall take the liberty of making the following points.

31. A CCC is Replaced by a CCC, but the new CCC isn't a CCC
What was a CCC is now a CCC, but the new CCC is significantly different from the old CCC, and this change will almost certainly have two very detrimental effects on any aggrieved household owner seeking redress for severe loss.

‘Code Compliance Certificates’ have been replaced with ‘Consent Completion Certificates.’

By no longer certifying compliance with the code, the Building Consent Authorities will almost certainly, and most happily, find that their liability for non-compliance of household type units will be significantly reduced.

Many serious defects are only discovered many years after construction, often near the ‘ten year cutoff,’ and unfortunately, often later.

The design and construction of even a single household unit is a lengthy process. The present liability of the BCA’s means that if serious defects are discovered years after practical completion, even though the designer’s defects may be time barred from legal action, the BCA’s liability for design defects is not, because the ‘ten year cutoff’ starts from the date the Code Compliance Certificate is issued.

By removing this code compliance certification, many more owners who discover very serious defects will be time barred from seeking damages from anyone.

I believe there will be a very significant reduction in the liability of BCA’s (local Councils) as a result of this very quiet, subtle change, further obscured by the retention of the same three letter abbreviation. The effect of this reduction in liability, and the further time limitations for claims that accompany it, will significantly reduce means of redress available to aggrieved owners, while the rhetoric surrounding this Act and the No. 4 Bill claims the opposite.

32. Sections 14B, 14D, 14E & 14F
These sections outline the responsibilities of the owner, designer, builder and building consent authority in respect of building projects.
It is right and proper they are described for completeness, but these responsibilities are in no material way any different to what they have been under common law and previous building legislation, since New Zealand became New Zealand, and going back to Britain, for centuries prior to that.

Even the Ancient Romans, even the Sumerians, would have regarded these responsibilities as ‘obvious.’

Anyone who reads this Act, or spoke in support of it, and thought that these sections somehow enhance household owner protection is misguided.

F. The Wider Picture

34. Building Consents

Building consents are not red tape.

Building consents do not make houses unaffordable. Housing unaffordability is largely the result of a pyramid/ponzi scheme fueled by near infinite amounts of fiat money, created out of nothing for the purpose of fueling this bubble, all aided and abetted by deliberately misaligned taxation.

Building consents are, until such time as all of the design and construction deficiencies within the NZ construction industry are eradicated, an essential measure in affording the future owners at least some reasonable protection from a disaster.

Leaving aside the responsibility of central Government for allowing the likes of untreated timber and monolithic sheet cladding to be forced on the public and forcing building consent authorities (in effect, Territorial Authorities, or ‘local Councils’) to accept them, many local Councils performed appallingly in giving Building Consents to sub-standard designs, and in approving unacceptable work.

However, as explained above, it was successive Labour and National Governments that deliberately destroyed the technical capabilities of these same Councils.

Despite that, I have personal experience, through carrying out structural reviews on behalf of BCA’s, of many ‘shockers’ that were averted as a result of Building Consent checks.

It is my experience, over many years, that the main cause of significant delays in the Building Consent process is deficient information being supplied by the applicant, not ‘Council bureaucracy.’ Usually, this deficient information is in the form of flawed design calculations, drawing deficiencies, or work not included. BCA’s are not responsible for this. This suite of legislation does nothing to address this primary cause of delays, but the rhetoric claims that such delays will be reduced.
Unfortunately, many Members of Parliament “still don’t get it,” and, having never designed, supervised or taken responsibility for a significant building project in their lives, want to revert to ‘the good old days’ of the early 1990’s.

One nonsensical feature of Building Consents imposed on local Councils is the ‘one time fits all’ consent period of 20 working days. Far from Hon Dr Nick Smith campaigning for “a late consent is a free consent,” he should have asked himself the following two questions:

“Why is it that a Building Consent Authority (BCA) has 20 working days to process the Building Consent application for a small beam in an existing house, yet the same BCA has again only 20 working days to process the Building Consent for a 60 storey skyscraper complete with 8 storey podium structure, a project at least 10,000 times larger than the small beam?”

“Why is it that a BCA has 20 working days to process the Building Consent application for a 60 storey skyscraper complete with 8 storey podium structure, but if subsequently, the owner of the skyscraper wishes to alter one structural beam, the BCA has 20 working days to process that consent application as well?”

With just about everything else in NZ building governance, it simply does not make sense.

35. **Productivity**

Aside from a minimum amount of capital needed for plant and the like, true productivity comes from two things and two things only – proper training and quality work.

Germany has these two essentials in abundance. As a result, German buildings are sound, and constructed very efficiently, with workforce productivity way above what we have. With respect to the wider economy, despite very high real wages, German maintains a massive trade surplus with the rest of the world, year after year after year.

Any delusions that the ‘building blocks’ that the present Minister and his department are putting in place will lead to a boost in productivity are easily dispelled for any open mind, with consideration of the following.

An engineering colleague of mine lectures part-time in structural engineering at one of the institutions that used to be one of those fine, practical technical institutes. Some of his first year students do not know how to work out the area of a circle. My colleague was recently called to account by the management, regarding a third year exam paper he set and marked. He was asked why he had failed half of the class, who were, after all, “paying fees.” “Because half the students simply weren’t good enough to pass,” he answered. Then he was criticised for presenting his questions in a realistic form, consistent with what is done in practice. That was simply too demanding, in the opinion of the management. Then he was criticised for taking too long to mark each exam of 8 questions – all of 25 minutes in fact. Such organisations used to produce world
class apprentices, but not any more. This decline in ‘technician’ training is not being addressed.

Especially for the essential professions such as architecture and professional engineering, formal training such as the BArch and BE degrees can only be the beginning of the training required to produce a truly skilled, competent, productive and confident building professional.

Unfortunately, the training of architects at the University of Auckland has gone backwards, especially since the leaky building crisis was exposed in 2002. NZ university trained architects used to be prized in Britain, because they could actually design a functional real building, unlike the graduates of the ‘leading’ British universities. The once stand-alone School of Architecture at the University of Auckland has been absorbed to be a department of the National Institute of Creative Arts and Industries. Students used to be required to do practical training in the form of carpentry and joinery, so they would have an appreciation of what was required to get a building actually built. I have been reliably informed that the last vestiges of such training were removed five years after the leaky building crisis was exposed.

Colin Nicholas retired from a very successful consulting engineering career to teach structural design in the University of Auckland School of Engineering. Because of the nonsensical assessment of university performance, which makes absolutely no allowance for the fact that professional schools must actually provide practical training and maintain existing knowledge, Colin has to carry the title ‘Designer in Residence,’ otherwise his department would be penalised for employing him to teach engineering students how to design buildings.

Dr Charles Clifton was for many years senior structural engineer at the Heavy Engineering and Research Association. No structural engineer could take more credit for the resurgence of multi-storey steel construction, such that steel now has over 50% of a sector it was once effectively excluded from.

Design procedures that Charles championed and developed, and explained clearly to practising engineers, resulted in the two best performing high rise buildings in Christchurch (refer Appendix H). For various technical reasons, these buildings were hit with earthquake loads well in excess of what they were designed for (unlike some other major concrete buildings, which failed despite only being hit with code level seismic loading), and suffered only very minor damage. Misalignment of the lift rails became the biggest problem.

Both Colin and Charles have increased both the quantity and quality of training in structural design in the School of Engineering. To the point where the students complain that they are being given too much work, work that is impinging on their other courses.

But the feedback Colin and Charles receive from their graduates is “You were right to pile the work on, because there is so much a structural engineer has to know,” and “We receive effectively no on-the-job training.” This lack of proper on-the-job training is not being addressed.
There are supposed plans to boost productivity in the building industry by 20% by 2020, all based on a document prepared after a talkfest, and presented to the Cabinet by the Minister as the plan for the way forward.

Following a ‘talkfest’ on the building industry in August 2008, to which only the ‘best’ people were invited, a committee, complete with various sub-committees, was set up to write a report to address the problem of low productivity in the construction industry.

The resultant ‘Report of the Building and Construction Sector Productivity Taskforce’ is nearly totally worthless (refer Appendix I). There is only one useful suggestion in 38 pages, contained in one paragraph, something I could have written in a few minutes. Yet this report took 11 months to write, despite the authors supposedly being experts in the field.

In fact, the authors of that report were senior members of the very companies and organisations that are the NZ building contracting sector. But if these people knew what was required to deal with the poor and declining standards of genuine productivity in the NZ building industry, why hadn’t they done it already, and solved the problem? Surely, as a matter of personal commercial interest, they would have enacted this training and reform throughout their own companies and organisations.

Eleven months to write 37 ½ worthless pages is real productivity, isn’t it?

As a result of the report, which was presented to the Cabinet as ‘the way forward,’ the Building and Construction Sector Productivity Partnership was established in 2010, with much the same people and organisations responsible for this report involved again.

What increase in productivity has this ‘partnership’ achieved in two years? I will be very surprised if it is anything other than zero.

As repairs are starting to get underway in earnest in Christchurch, there are reports of a shortage of semi-skilled workers like painters and plasterers, and almost immediate calls for them to be imported from overseas.

In the 15 ½ months since the 22 February 2011 earthquake, couldn’t this ‘partnership’ have at least trained a few unemployed people to properly use a paintbrush?

36. Additional Examples of Ministerial and Departmental Incompetence
(a) In 2005, the IPENZ Board agreed to set up my Taskforce, which would have had NZ’s genuinely top engineers on it for the purpose of dealing with the crisis within the structural engineering profession. Between the decision to set up the Taskforce, and its secret subversion by the then Deputy President, Peter Jackson, and the Chief Executive, Andrew Cleland (a food technologist), I had two meetings with the DBH.

I stated clearly that welders needed to be added to the proposed list of Trade Licensing Classes. Welders are critical to the safety of many large buildings,
especially stadium type structures, but the DBH seems incapable of even considering any building other than a house or town house. Of course, I was ignored.

Appendix J contains a copy of Building (Designation of Building Work Licensing Classes) Order 2010. Refer to page 4, which contains the Trade licensing classes. You will see that Carpentry, Roofing, External plastering, Bricklaying/blocklaying and Foundations are covered. Welding is not included.

A major contributory factor to the collapse of Stadium Southland, a collapse that could have occurred with thousands of people in the building, a collapse that could have killed and injured dozens if not hundreds of people, was appalling standards of site welding.

(b) Page 3 of the same document lists a ‘Design’ licensing class, responsible for the design of a range of building ‘categories,’ from 1 to 3.

This document is confusing, but the intent, and I don’t think it has changed, was that ‘simple’ Category 1 buildings, such as a very straightforward house, could be designed by a Design Draftsman, while Chartered Professional Engineers would be required to design ‘complex’ Category 3 buildings. An undefined intermediate type of structural designer could design the intermediate Category 2 buildings.

But the drafting of this document, a document fully approved by the current Minister, is so incompetent that a vast array of extremely important buildings, which all common sense demands be designed only by very competent structural engineers, will not have to be designed by Chartered Professional Engineers.

The definition of a Category 3 building is so incompetent, that numerous critical structures are excluded.

Refer to pages 7 to 8 in the document.

A Category 1 building is essentially a single house, with a ‘low risk of leaks’ and not too exposed to the wind.

A Category 2 building is neither a Category 1 building nor a Category 3 building.

A Category 3 building is defined as one that is not a Category 1 building, not a single house, and whose height equals or exceeds 10 metres.

But the definition of building height is completely incompetent.

Building height is defined in one way and one way only:

‘building height means the vertical distance between the upper surfaces of the floors of the building’s lowest and highest storeys’
and:

‘floor does not include a mezzanine floor or a rooftop area’

(their bold, my italics).

In other words, to be classed as Category 3, a building must have at least two floors. But numerous critical structures, which absolutely require a highly qualified engineer to design them, have only one floor, or none at all.

Ignoring internal sub-structures, major stadia such as the Vector Arena and Stadium Southland only have one floor. Nearly all of the large buildings at NZ Steel, which support crane girders that in turn support cranes that lift 100 tonnes and more, have only one floor. Some external crane support structures at NZ Steel have no floors, and are less than 10m high. What about wharves and extremely tall communications towers?

If (God forbid) we had a nuclear power station, many of the most critical buildings, including the main containment structure, would, according to this nonsensical document, not need to be designed by a Chartered Professional Engineer. Some sort of ill-defined intermediate designer would do.

(c) It only took the DBH about five years, and the expenditure of many millions of taxpayers’ dollars, to arrive at the point where they were capable of writing the aforementioned Order in Council, a document that clearly, is not worth the paper it is written on.

(d) Further to this multi-year, multi-million effort by successive Ministers and the DBH to implement the Licensed Building Practitioner scheme, the DBH recently released a mandatory document called ‘Memorandum from licensed building practitioner: Certificate of design work.’

The purpose of this document is supposedly to ensure that an appropriate Licensed Building Practitioner has designed restricted building work, *for a particular project*.

A copy of the form as it was released for use is included in Appendix K.

As soon as it was released, the NZIA, IPENZ and ACENZ immediately told their members *not to sign this form*, because of the wording of the final Declaration.

This despite the DBH supposedly working in close cooperation with NZIA, IPENZ and ACENZ.

Subsequently, the DBH issued a statement that alternative wording of this Declaration, as advocated by NZIA, IPENZ and ACENZ, was acceptable.

But when I went to fill out this form for the first time, I realised there is no space on it for the most critical information, something that no one else had commented on.
There is absolutely no space assigned for a description of what the particular project entails. This space should be at the top of the first page.

Instead, the front page allows for the address of the site, and the details of the owner.

But at any one address, a project could entail a completely new house, or the addition of a second storey or a new wing, or a separate sleepout, or just internal modifications to an existing house.

By accident, there is some space on pages 2 to 4 where the nature of the project could be repeated beside each element of restricted work, but that is not the way it should be.

Just like the ridiculous Order in Council for licensing classes described above, the DBH laboured long and hard to produce this nonsense, and the poor taxpayer has to pay for this!

This has to stop. Root and branch reform of the building and construction industry, including the legislation, regulations and authorities that govern it, and the educational bodies that support it, is desperately needed and years overdue.

Soon, it will be too late to turn the situation around, even with all the will in the world.

The completely misleading ‘tinkering at the edges’ embodied by the Building Amendment Bill (No 4) does not provide this essential reform, nor is it a stepping stone to more comprehensive reform.
Appendix 13

Failed Timber Dance Floor
University of Canterbury
2012
Jumping collapses venue floor

SAM SACHDEVA
Last updated 12:32 14/07/2012
Share

TOMMY ILL

Hip hop concert opening act Tommy Ill said the floor collapsed under the weight of the crowd.

Structural engineers will investigate why the floor of a new $2.5 million events centre collapsed during a Christchurch hip-hop concert.

A Canterbury University concert was abandoned last night after part of the floor at its new multimillion-dollar venue collapsed during a performance.

The concert, The Perfect Storm, was held at the university's new $2.5 million temporary events centre.

Designed by Christchurch architecture firm Warren and Mahoney, the centre opened in April after the university's original student bar was damaged in the February 2011 earthquake.

University spokesman John MacDonald said the university would consult structural engineers to determine the cause of the collapse.

"We are very thankful that no one was injured. The safety of our students is paramount, and we will need some time to determine what has happened and what we are going to do about it," MacDonald said.
MacDonald said there were about 900 people in the events centre area at the time, well below the building's capacity of 1400.

The events centre would remain closed over the weekend while investigations continued, he said.

A concertgoer, who did not want to be named, said the floor gave way just after midnight during a performance by hip-hop artist Savage, due to the "sheer weight" of people.

"You know how crowds push to the front: there was just too much weight on the front, then everyone jumped and it just gave out."

While most people were annoyed that the concert had to be called off, the man said it could have been far worse.

"A metre to the left and the stage would have gone through into the ground; thank God nobody was hurt."

The concert was put on by the University of Canterbury Students' Association (UCSA) as part of a re-orientation week.

The association's Facebook page has been filled with comments about the collapse, with some students asking for a refund and questioning the safety of the venue.

"I cannot understand how a floor collapses when you are suppose to be engineering a building for a huge ***** earthquake, sort your ****," one person said.

UCSA president Erin Jackson said the association was currently examining reports from the night and speaking to university officials to determine what had happened.

Jackson said nobody was injured during the performance and everyone was safely evacuated.
Appendix 14

Condemned Apartment Building
on Park Terrace, Christchurch
Condemned Apartment Building on Park Terrace, Christchurch, 2011

Note complete lack of walls or bracing to this western face opposite Hagley Park
Appendix 15

Scarry/Clifton Submission on Draft Central City Plan for Christchurch, Sept. 2011
Comments on the Draft Central City Plan for Christchurch
by John Scarry CPEng and Dr Charles Clifton FIPENZ, FNZSEE

1. Executive Summary
This submission from two leading structural engineers seeks to raise awareness that:

- The draft Central City Plan and the desired redevelopment it proposes have serious implications from a seismic perspective. Very sound engineering advice needs to be sought, carefully considered, and the Plan needs to be modified accordingly before being adopted as official policy.

- Far from tall buildings *per se* being dangerous in earthquakes, two of the best performing buildings *of any height* in the recent earthquake were a 12 storey steel framed and a 23 storey steel framed building. Poorly designed and constructed buildings, irrespective of height, are what is dangerous.

- Although soils performance was generally good within the confines of the CBD, good practice would lead to the peat that is prevalent in the top 5m of soil being removed and replaced with compacted hardfill, and vibrocompaction equipment being used to compact the top 20-30m of the underlying gravels and sands. This soil improvement cannot be done near most existing buildings, and this type of soil improvement is much more cost effective the taller the building is.

- Timber as it is presently available in New Zealand, and as it must be used in most modern buildings, is not the ‚eco-friendly,‘ sustainable material it is purported to be. As explained below, upon demolition of the building, most timber is only fit for landfill, unlike steel and concrete, which can be and are recycled.

2. The Submitters

**John Scarry** BE (Hons), ME, CPEng is a consulting structural engineer from Auckland with 28 years in the construction industry. His revelations about poor practices in the structural engineering profession and construction industry were, along with the revelations by the O’Sullivan brothers regarding rotten timber, responsible for the drafting of the Building Act 2004.

His particular fields of interest are professional practice and governance, the interaction of various parties in the construction industry, and the often massive
problems caused by apparently minor defects in legislation, codes or accepted practice, particularly as they affect sound seismic design.

Charles Clifton BE (Hons), ME, PhD, FIPENZ, FNZSEE is Associate Professor of Civil Engineering at the University of Auckland. Charles has 32 years’ experience in consulting engineering, research, teaching and the development of structural standards.

He is an internationally recognized expert in the performance of steel structures under seismic and fire loading. His other fields of interest are corrosion, composite action, light gauge steel framing and the acoustic performance of buildings and building elements.

Charles is a senior advisor to NASH, the National Association of Steel Housing. He is a member of the Department of Building and Housing Working Groups on aspects of fire safety design and acoustic performance.

He has developed several new forms of „low damage’ seismic resistant systems.

Charles is a Fellow of the Institution of Professional Engineers New Zealand, a Fellow of the New Zealand Society of Earthquake Engineering, and on the Management Committee of the Society of Structural Engineering New Zealand.

In his role as a member of the Standards New Zealand Committee on the Loadings Standard, he is involved in setting seismic design loads for buildings and structures of all material types.

Charles is Chairman of the Standards New Zealand Committee on the Steel Structures Standard.

Charles has been responsible for investigations into numerous failures in buildings, especially buildings in which crowds gather, and advising on repair solutions.

We do not wish to appear at the public hearings, but will answer any questions you may have.
3. **The Purpose of Our Submission**

The draft Central City Plan describes an admirable attempt to create a varied yet harmonious, pleasant and sustainable living and working environment within the rebuilt central city area of Christchurch.

Yet the rebuilt central city area must be safe from a seismic perspective, in fact, it is the stated goal of Mayor Bob Parker that the rebuilt Christchurch be “the safest city in the world, from a seismic perspective.”

And not only safe. It must be resilient, with minimal damage in severe seismic events so that the disruption that has occurred is never repeated.

But the structural design of seismically safe, low damage structures is not something that can be added on as an afterthought, once the land ownership, planning, business use, living and architectural requirements have been accounted for. Seismic resistance must be integral from the start, and that includes the very basis of the Central City Plan.

Based on our vast experience, we can see in the proposed concept for the central city rebuild very serious potential conflicts and problems if seismic safety and damage minimisation are to be achieved. These conflicts will result in disgruntled owners and occupiers, much wasted building space or seismically deficient structures, or, most likely, a combination of all three.

We ask that the Central City Plan be revisited so that the implications of what we warn of are fully understood, and fully allowed for, and any adverse consequences of certain schemes clearly stated in the Plan.

We also wish to clarify some serious misconceptions regarding the performance of tall buildings in earthquakes, and the „sustainability” of timber as a building material, and provide some useful information on the effect of form on the insulation of building in cold areas, and the soils in the central city.

4. **“They’re Only Sketches and Conceptual Images”**

We are fully aware that the images in the draft Central City Plan – Volume 1 („Volume 1”) are only sketches and conceptual images, but that is irrelevant.

They convey perfectly adequately the type of development that is being proposed, and what the public are „buying into.’

For example, the images on pages 49 and 50 of Volume 1 (included in the attached Appendix A) show a street full of three to five storey buildings built in close proximity to each other on adjoining lots. Many of the buildings have retail or similar space on the ground floor, and apartments above. There is considerable variation between the buildings – they are all different, but complementary. To give openness on the street side, beams and columns provide transverse seismic resistance.
Uniform frontages, with all buildings having exactly the same storey heights, and wide walls or diagonal bracing in the transverse direction are not mandated for every building.

It is our knowledge of what is required to design buildings of the type shown on pages 49, 50 & 105 of Volume 1 to properly resist large seismic forces, and the effect of fire regulations and property rights, that make our comments absolutely relevant, and it is essential that they are given full consideration.

Otherwise, the central city rebuild will result in disgruntled owners and occupiers, much wasted building space or seismically deficient structures, or, most likely, a combination of all three.

The people of Christchurch need a plan that has proper seismic resistant design built in from scratch, and the people of Christchurch have to be made fully aware of the constraints this imposes. For the conflicts of maximizing floor space, ‘architectural freedom’ and proper seismic resistance to have to be revisited for every new building, ad infinitum, is simply not acceptable, and the reality is that it will be proper seismic resistance which will suffer the most.

5. **The Implications of What is Planned**

(a) **Seismic Gaps Between Buildings**

Please refer again to the concept sketches on pages 49-50 of Volume 1 in Appendix A, and the two photographs in Appendix B of a real, three storey residential building under construction in Christchurch.

These images are taken from what we will call the front of each building. We will call the direction perpendicular to the footpath, down the depth of the buildings, as the **longitudinal direction**. The direction parallel to the footpath and the front of each building we will call the **transverse direction**.

Each building is taken to stand on its own discrete parcel of land.

The building concept that is shown on pages 49-50 of Volume 1 could be applied in London, Paris or any other non-seismic city without further consideration. But earthquakes require further consideration.

It is an absolute legal requirement that every building must have full height firewalls, separating it from adjacent buildings, down the entire longitudinal length of the building. These walls will usually be reinforced masonry or concrete, and for the height of buildings involved, they should be 190mm to 200mm thick.

It is an absolute legal requirement that these firewalls must stand completely **within** the parcel of land the building stands on.

But it is also an absolute legal requirement, through the Building Code, that the outside face of these firewalls must be sufficiently set back from the boundary...
(except to the street) that they cannot cross the boundary even under the large deflections caused by severe earthquakes.

The reasons the entire building structure, including the firewalls, must not cross the boundary are that neighbouring buildings tend to vibrate out of phase, so they can move towards each other, and pounding of buildings when they smash into each other in an earthquake is a catastrophic failure mode that must not be allowed to occur. It can easily result in collapse of one or both buildings, especially when the floor levels are different between buildings.

For reasons we will expand on below, the type of „open” building layouts shown on pages 49-50 mean that completely safe buildings will still deflect horizontally approximately 1% of their height, in an extreme seismic event.

(Precise calculations can and will be done for each real building, but this is a very reasonable estimate of the deflections involved).

For a 15m tall building, this means that the firewalls must be at least 150mm clear of the boundary. For adjoining buildings, that means that each building will lose floor space equal to 0.3m width times the full depth of the building.

It also means that there will be 300mm „alleyways” between buildings that have to be effectively sealed off or may have problems relating to rubbish, feral animals or vermin. If human access is required for occasional cleaning or maintenance, 500mm gaps may be required.

These seismic separations must be rigorously enforced, without any „tweaked” calculations or „loopholes” that allow inadequate separation.

Now there is absolutely no problem in losing floor space and having 300mm or more gaps between buildings, but we believe everyone needs to be made completely aware of these requirements in the plan, without confusion, and all relevant conceptual drawings and the text of Volume 1 need to clearly show this.

(b) Conflicting Requirements Between Floors, and Potential for Soft Storeys
Given the street frontage, and the „tunnel” nature of closely spaced buildings with complete fire walls down two sides, there is an obvious desire to have the end walls and interior space as „open” as possible. Also, the requirements for „open” retail space and freedom for the layout of apartments combine to mean that in most buildings, transverse structural walls or bracing to resist seismic forces will be unacceptable, and transverse lateral resistance to seismic loads will have to be provided by rigidly connected beams and columns, forming what are known technically as moment frames.

Except when the members are extremely large, moment frames are inherently more flexible than walls or bracing under lateral loads, hence the issue of relatively large horizontal deflections under extreme seismic loading raised above, and the requirement for large seismic gaps between buildings.
The conceptual drawings on page 49, 50 and 105 of Volume 1 attempt to show a mix of different architectural forms between the ground floor space and the apartments above, and within the apartments themselves. Similarly, set back balconies, differing between floor levels in some instances, are shown to make the whole concept more interesting and appealing to the public.

However, the reality is that in order to provide structures with excellent dependable seismic performance, a far greater degree of uniformity will be required. Without it, the inevitable result will be ‘compromises’ in which the seismic resistance will suffer, and in many instances, buildings with catastrophic ‘soft-storeys’ due to deficient structure in the ground floor will result.

Even if the additional cost is ignored, accommodating inset balconies with the required slab set downs, offset walls and offset columns, all consistent with excellent seismic performance may simply not be feasible, because of the storey height limitations.

Set back balconies of the type shown on page 105 of Volume 1 render proper insulation in a cold climate impossible, yet the Plan does not seem to appreciate this fact.

Excellent seismic resistance must be built in from the very start, and is not something that can be fitted in once every other requirement has been met. And the Central City Plan must reflect the form of buildings that must be produced if the goal of having a very seismically safe city is to be met.

Don’t forget that the old, mainly two storey buildings that collapsed in the CBD were completely incompatible with good seismic resistance from the start, because they followed certain building fashions and were built at a time when seismic resistant design was not understood. This mistake cannot be made again, especially when we are now dealing with five storey buildings, with four heavy suspended concrete floors.

With reference to Appendix B, Photo 1 shows the ‘tunnel’ form. Whereas this low rise development may end up with sufficient braced plywood panels in the transverse direction to be seismically safe, imagine the same form with three more storeys of apartments on top, all with concrete floors, and the conflicting layouts when trying to get strong moment frames down to the ground.

With reference to Photo 2, where is the seismic separation to the neighbouring building? It probably won’t matter much in this instance, because the neighbouring building appears to be a ‘leaky’ timber framed structure, so will just crumble locally under pounding, but you get the idea.

(c) An Innovative Alternative
One alternative that could be proposed in the Central City Plan, to provide transversely stiff buildings and eliminate much of the lost space due to double fire walls and seismic separation, would be to promote the amalgamation of perhaps three to four adjoining properties, upon which a single structure would
be built, with „unit titles’ allowing multiple retail and apartment occupants. All floor levels would have to be consistent throughout, however. The necessary transverse bracing or shear walls could then be concentrated in one area, where the retail and apartment layouts could accommodate it.

This approach has many benefits. It would halve the number of fire walls required, eliminate many seismic separations, and with regards to foundations, eliminate much of the costly offsets and duplication of piled foundations described in Section 9.

6. **Tall Buildings Are Not Dangerous In Earthquakes**

Tall buildings are not dangerous in earthquakes. However, badly designed and constructed buildings are dangerous in earthquakes, irrespective of height.

Tall buildings are not inherently ugly. Ugly buildings are ugly, irrespective of height.

Tall buildings offer many advantages with regards to making soil and foundation improvements. They make extensive soil improvement far more cost effective. If properly planned, they allow plenty of open space around them, whilst providing the same density of occupancy in the CBD as more closely packed low rise buildings. Well spaced, they can also completely eliminate the risk of pounding between buildings in earthquakes, and the need to provide seismic separations.

The two worst performing buildings in the 22 February 2011 earthquake were the six storey CTV building, with 115 dead, and the five storey Pyne Gould building, with 18 dead.

These low rise buildings, one „modern’ and one „pre-modern,’ in seismic design terms, are of a height that is not only permitted under, but actively encouraged by, the draft Central City Plan. Yet their collapses accounted for over 70% of the total death toll in the February earthquake.

The draft Central City Plan would ban the construction of two of the best, if not the best, seismically performing buildings, the 23 storey Pacific Tower and the 12 storey Club Tower (refer Appendix C).

These are ordinary residential/commercial buildings, with eccentrically braced steel frames, and are both less than 3 years old. For various technical reasons, they were subjected to seismic loads far in excess of what they were designed for. All structural yielding was confined to where it was supposed to be. Even then the effect of yielding was negligible, and both were capable of immediate re-occupation, although the lift rails need re-alignment.

The draft Plan would ban the construction of these buildings. Only low rise buildings can be built, supposedly to ensure seismic safety. But that is like saying that one tonne of lead weighs more than five tonnes of feathers.
(With reference to Appendix C, it appears that, despite the Club Tower having 12 dangerous storeys, CERA and the Christchurch City Council are happy to be tenants!)

What could be described as „the first building of the rebuild” is shown in Appendix D. This article from The Press on 16 June 2011 shows a brand new building, of the type the draft Central City Plan proposes for new shops in the CBD, leaning over at 5° after the moderate earthquake of 13 June 2011. This building has since been demolished.

Imagine buildings like this stacked up against each other, block after block, street after street. Not only that, but with three or four storeys of apartments on top.

Many modern high rise buildings have failed because they were poorly designed and/or poorly constructed, not because they were high rise buildings.

It is only a slight extension of the „logic” behind the „seismic safety” concerns in the draft plan to ban the reconstruction of all cathedrals and „old style” church buildings, because they have all been severely damaged in the earthquakes.

The fundamental requirements for seismic resilience are proper form, proper design, and proper construction, all combined with common sense and freedom for the engineer to start with a sound structural form.

Ban high rise buildings from an aesthetic perspective, if that is what is wanted, but do not ban them on the grounds of seismic safety.

With regard to page 55 of Volume 1, and the taller buildings that will be allowed in certain circumstances, the top 45° bevels required for the top storeys are very bad from a seismic perspective, introducing additional torsion into the building. The recent earthquakes have shown that symmetrical buildings perform far better than similar type buildings that are asymmetrical, even when the engineers have made a reasonable attempt to deal with the torsional effects.

The effect of these setbacks on seismic performance must be stated clearly in the Plan.

7. **Good Foundations**

Despite liquefaction being used as an early excuse for much of the poor performance of buildings within the CBD, apparently most of the CBD was spared serious liquefaction.

However, it would be prudent to improve the underlying soils where existing buildings do not prevent it. Such improvement would include stripping out peat layers in the top 5 metres, and vibrocompacting the gravel and sand layers down to a depth of 20 to 30 metres. Refer Appendix E for some diagrammatic information on vibrocompaction.
Such ground improvement is much more economical the taller the building is.

However, it cannot be done near existing buildings, especially vibrocompaction. Page 115 of Volume 1 shows existing buildings having new mid-rise structures being built next door to them. How can the ground for these new buildings be improved? What thought has been given to this?

8. Basements
We could not see that basements have been banned, but they must be. The near universal flooding of basements in Christchurch due to the granular soils and the very high water table mean that, except in the most exceptional circumstances for buildings of the utmost public importance and with fully tanked basements, all new basements below the water table must be banned.

9. The Relatively High Cost of Foundations for the Proposed New Development
Given the intent of the Central City Plan to replace what were largely two storey buildings (containing one suspended timber floor) with four and five storey buildings (containing three and four suspended concrete floors), what consideration has been given to the costs of the foundation systems required?

Unless extremely good soil improvement is achieved, the new buildings will have to be supported on piled foundations. The practicalities of constructing piles near existing buildings mean the piles will have to be set back at least 1m from the boundary, but the requirement to locate columns above as close to the fire walls as possible mean that large foundation beams will be required to transfer the resulting offset.

Such foundation beams are actually very worthwhile even if there is no offset, from a seismic performance point of view, but the maintenance of discrete buildings on separate lots of land mean that there will be two rows of piles from adjoining buildings in close proximity. The innovative alternative of combining titles over three to four lots to provide one structure would allow one row of piles under each internal fire wall between unit titles.

10. Timber – The Sustainable Material?
The draft Central City Plan promotes timber as the „sustainable’ building material. It is simply not the „super sustainable’ material it is purported to be.

If we built dwellings in the way they were up to the Middle Ages, using large sections of hardwood and old growth pine, connected with hardwood pegs and completed with wattle and daub and the like, then timber could be considered a truly sustainable building material. Once a building was demolished, the timber could be used to make utensils and furniture, or be burnt, or composted.

But in modern New Zealand, it is a reality that the bulk of the construction timber we use is small size, it must be treated (and until the condensation issues
within buildings are properly dealt with, it should be treated to a minimum of copper-chrome-arsenic H3.2), it is full of nails and connectors, and is covered with often toxic paints, varnishes and adhesives. Upon demolition, such timber must be put into landfill, whereas steel can be, and is, recycled *ad infinitum*, and concrete can at least be crushed and used as hardfill.

Yours faithfully,

John Scarry  BE (First Class Honours), ME, CPEng

Dr Charles Clifton, BE (Hons), ME, PhD, FIPENZ, FNZSEE
Appendix A

Extracts from draft Central City Plan
A distinctive city

Distinctive City provides the framework from which a well-designed Central City will develop.

Cities are complex and diverse and there is no one project that can deliver this or a single outcome or style that can be specified. The best cities are dynamic, vibrant and creative. They evolve over time in response to strong community leadership and their environment, as well as taking advantage of new opportunities that arise.

The Central City is greater than the sum of its parts. The private sector with the assistance of designers, engineers and the construction industry has a responsibility to develop individual parts of the Central City that contribute to the greater whole.

A vision for the Central City is represented in the typical streetscape (below).

A series of initiatives are outlined in this chapter that will help guide private investment in collaboration with the Council. These initiatives inform a new regulatory framework (see Volume 2) that will seek high quality outcomes. To successfully deliver this a strong partnership between the public and private sector is required.

Below: A low-rise City respects surviving heritage and creates a rich and vibrant ground level experience. Building heights are at a level where people feel comfortable in and around, and integrate with the streetscape environment.

New lower building heights are proposed for the Central City following the earthquakes.

Christchurch’s redeveloped city centre will contain well-designed lower rise buildings and public spaces that create an identity for Christchurch, different from that of other large centres in New Zealand.

The earthquakes have had a devastating effect on buildings in the Central City. Many tall buildings, which have helped define the city’s skyline in the past, are damaged and likely to be demolished.

While a few tall buildings will remain, and will need to be integrated into the redeveloped city, the majority of the remaining buildings in the Central City will be lower rise.

A typical Central City street scene:
Strong and resilient buildings

The community have asked for a safe Central City. The lessons of recent events must be applied so that the city takes this opportunity to rebuild a place that is safe and resilient and leads the practice for New Zealand.

Confidence in the strength of buildings in the Central City is essential for people to return and reoccupy buildings and spaces. Lower building heights and adherence to the Building Code standards will assist in helping people feel safe. The Council will process building consents in line with the Building Act and will review its Earthquake-prone, Dangerous and Insanitary Buildings Policy.

A Royal Commission of Inquiry into building failure as a result of the earthquakes will report no later than 11 April 2012. The Royal Commission will inquire into the performance of buildings within the Christchurch CBD and the adequacy of the current legal and best practice requirements for the design, construction and maintenance of buildings in central business districts throughout New Zealand to address the known risk of earthquakes. It is anticipated that lessons from the inquiry will need to be implemented at the local and national level.

The Council will encourage building owners to exceed the code requirements through the standards achieved for new buildings and retrofitting of existing buildings. Recognition for buildings that meet or exceed the building code levels will be considered. This could take the form of a certificate for display or other visual indicator at the entrance of a building that the appropriate measures have been taken and the public are entering a safe building.

The strength of buildings and the safety of the Central City will form an important part of the city’s future identity.
Height and human scale

A lower-rise Central City will have greater variety and more consistent density of activities and buildings to maintain the overall capacity in the area. The scale of remaining heritage buildings will be respected.

One of Christchurch’s distinguishing features has been the spire of ChristChurch Cathedral. It was an important reference point for Christchurch before the earthquakes. Mature trees throughout the Central City will also form an integral part of the city’s rooftopscape and, together with lower building heights, form the basis of a new identity.

Lower building heights will moderate the city’s notorious winds and let in greater levels of sunlight to create better public spaces. The vertical scale of the streets becomes more people-friendly as the dominance of taller buildings diminishes. People will interact more at street level, creating a vibrant city environment.

Lower heights will let more sunlight into streets and public spaces and provide a people-friendly scale to the Central City. These also contribute to people’s perception of safety as building occupants can maintain a connection with the street, by recognising faces or hearing sounds. People in lower buildings can more easily wander outside and contribute to street activities, helping to create a vibrant environment.

There are differing perceptions of what building height and how many storeys may be considered low rise. This plan outlines an approach which responds to the community sentiment of a low-rise city, creates liveable streets and spaces, and is mindful of the economic realities and demands for future capacity in the Central City.

Minimum and maximum heights will be identified in the City Plan. Heights will vary from the Core (see maximum building heights across the city) with a maximum of seven storeys to the Edge with a four storey maximum. An additional floor will be offered as an incentive where good urban design and green technologies are employed. Heights along street edges will be restricted to achieve a good street environment with additional storeys stepping back.

The heights of some key streets and places will be lower to provide for increased sunlight and recognise the character and sensitivity of these edges, for example City Mall and Cramner Square.

Variation of heights across the Central City:

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<tr>
<td>Cathedral</td>
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<tr>
<td>Core</td>
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<td>Edge</td>
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Height and human scale project

**What:** Minimum and maximum height limits with additional floors as an incentive where good urban design and green technologies are employed

**When:** Once the Central City Plan is adopted

**Who:** A shared responsibility between building owners, designers, engineers, and local authorities

Left: A gradual reduction of height from the Central City core to the edge. This will respect the visual connection to the spire and the height of trees. There will be an option for a bonus floor.
Compact CBD

A compact central business district is vital for economic prosperity in the redeveloped Central City.

Historically, Christchurch’s central business district has been too large for the city’s population and number of employees. This has resulted in the uneconomic use of prime real estate and ad hoc development.

A compact business core, bounded to the north and west by Avon River/Ōtakaro, Lichfield Street in the south and Manchester Street to the east, will provide for better economic growth in the long term and greater certainty for property owners.

Central business districts are important for delivering an economically and socially vibrant city. Compact central business districts enable more frequent meetings and exchange of ideas, easier access to services, and better provision of infrastructure, along with better social opportunities for employees. They have easier access to new living options within the neighbourhood in which they work, along with cultural activities and plenty of opportunities for social interaction. Ultimately, these areas support and enable higher economic growth for the region.

Delivering the compact central business district requires incentives with a spatial effect, regulation and the certainty delivered by the Central City Plan. Implementation of the Compact CBD will include coordinated changes to public transport facilities, transport access corridors and pedestrian areas. This is addressed in the Transport Choice chapter.

This project aims to encourage development in the central business district to create a high-functioning, high-density business and retail district. While office and retail development will continue to be widespread, the Council will actively incentivise and regulate to encourage the creation of a high-value and high-energy retail and office core - and distinguish this from other supporting retail and office area.

Council-led urban design guidelines and public investment in people-friendly streetscapes will help to visually define the area. The majority of the compact central business area land is owned and will be developed by the private sector. To accelerate development and attract new tenants, a range of proposed incentives will be available for development, redevelopment, and retail and office relocation to the compact central business district. The full proposed incentives package follows.

The investment in the public realm will be ongoing, while the incentives will be reduced as development and employed targets are reached.

“CBD needs to be less spread out, so it feels lively and is easy to get around. Also need to do something to attract people back from malls.”

Marjorie, Christchurch
Transitional City

Transitional City will ease the rules and barriers for a temporary period allowing the market to test ideas, explore concepts and develop innovative ways to bring people, business and investment back into the CBD.

It is a project-led approach to sustain the spirit and ensure the successful transition of the Central City as it redevelops post-demolition through a variety of temporary activities to become the city asked for by Greater Christchurch residents and key stakeholders during Share an Idea. There will need to be a strong alignment of all temporary activities to support the redeveloped city which will evolve during the next 10 to 20 years. The Transitional City project is critical to the success of the recovery of the Central City. A co-ordinated response from public sector agencies, including the City Council and CERA, will be needed to manage and facilitate the transition of the Central City. This coordination will be achieved by establishing a Council-led project team to centrally capture and coordinate a response to the broad range of quick win initiatives designed to accelerate the redevelopment of Christchurch. These projects will address a broad range of opportunities, ranging from attracting business and investment to the city, through to innovative ideas for achieving positive social, cultural and economic outcomes.

The Transitional City project will deliver a consistent response to all ideas and opportunities, as well as maximising the understanding of market demand and opportunity, allowing quick wins to be progressed efficiently and effectively. The project plan will be developed by the Council and will address governance, management, resources, funding, and provide clarity around the project scope, objectives and key performance indicators. Incentives will be necessary to achieve growth as the Central City recovers; a range of these are proposed across the various chapters.

Key themes

The Transitional City Project is comprehensive across the following themes:

**Green City**
- Greening the Rubble
- Avon River Park/Papawai Ōtakaro
- Central City Greenway
- Pocket Parks
- Community Gardens

**Distinctive City**
- Remembering - fabric drops on landmark sites and site hoardings
- Lighting key areas and specific
- Celebrating milestones
- Social and community services
- Demolition grandstand

**City Life**
- Gap Filler
- Community events
- Temporary art activities
- Public art programmes
- Temporary libraries
- Cardboard Cathedral

**Transport Choice**
- Temporary bus exchange
- Road access plan - CBD
- Temporary passenger transport shuttles
- Temporary cycle lanes and streets closures

**Market City**
- Retail Strategy
- Quick wins initiative
- ReStart
- Covered Market
- Farmers Market
- Expo Market
- EPIC
- Events Village
- Temporary Tourist Information Centre
- Free Car Parking
- Restrictions on Suburban Development
- Central and Local Government commit to return to CBD
- Property Development 101

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**Transitional City project**

**When:** Project Team framework and co-ordination currently under discussion with Council and CERA, expected to be finalised by the end of August 2011.

**Where:** Central City (four avenues) with priority focus on newly defined Compact CBD, entral City (four avenues) with priority focus on newly defined Compact CBD

**Who:** Council led with governance support from CERA and from other agencies, such as Canterbury Development Corporation, as appropriate.

**Cost:** Project management and operational costs $3 million over 10 years; Green City (transitional) $2.7 million over four years; Distinctive City (transitional) $5 million over four years; City Life (transitional) $250,000 over five years; Market City (transitional) nil, and Transport Choice (transitional) $3.1 million over three years.
Appendix B

Photo 1

Photo 2
Appendix C

Case Study: Pacific Tower
23 storey mixed EBF and MRF, composite floors, transfer diaphragm level 6
- Building has effectively self centred:
  - 60 mm out of plumb midheight
  - 30 mm out of plumb at top
  - under 0.1% residual deflection
- Minimum damage
  - No structural or non structural repair or replacement needed
  - Requires only realignment of lift guide rails
  - Could re-occupy but is in red zone so no public access
  - All other buildings of same height severely damaged; replacement likely

Case Study: Club Tower
12 storey mixed EBF and MRF, composite floors, torsionally irregular
- Building has effectively self centred:
  - 45;35 mm out of plumb top; within construction tolerances
  - 0.14% maximum residual deflection
- Minimum damage
  - Lift guide rail realignment required: this will cost approx $250k
  - No other structural or non structural repair or replacement needed
  - Building now fully occupied including CERA and CCC
  - The only normal importance high-rise building in Christchurch now reoccupied (other is Police Station)
DEJECTED: Michelle Crouch wants to know why her new building performed so poorly in the earthquake.

**The owner of one of Christchurch's newest building is demanding to know why it failed in Monday's quakes - just a day before it was to open.**

MG Nails and Beauty modern purpose-built tilt-slab premises on Barbadoes St will have to be demolished after Monday's 6.3 magnitude quake, with part of the ceiling caving in and the walls shunted into a precarious angle.

Business and building owner Michelle Crouch said when it struck, two contractors completing the finishing touches inside were lucky to escape unscathed as part of the building collapsed. She believed if the quake had come a day later, when the shop was full of customers and staff, it could have killed someone.

Crouch is demanding to know how a new building signed off by builders, architects, engineers and the Christchurch City Council could perform so poorly in the quake.

"I want answers because I don't want this to happen to anyone else. If my girls were in there how would I be able to live with myself," she said.

However, the building's engineers said the buildings had performed above expectation under exceptional strain, staying upright and allowing the occupants to escape.
Crouch said she had sunk $400,000 into the building and while insurance should cover the loss she was unsure whether she could bring herself to rebuild.

"I'm not putting anyone down I just don't have any confidence in this sort of thing anymore. I don't even know if I want to rebuild."

She said she had warned others to stop and think before rebuilding their premises after the quakes. It is the second time Crouch has lost her business.

Her Fendalton beauty parlour was destroyed in the September quake.

She has been operating from her home since and was looking forward to restarting her business in a safe new modern building, she said.

The building has been under construction since before the September quake after Crouch demolished the previous building which was deemed too old to economically quake strengthen.

She claimed she was told by engineers the new building would be strengthened beyond the building code requirement, designed to withstand not only the impact of a strong earthquake but the possibility of a neighbouring building collapsing onto it.

After the February quake the building sustained some damage.

But it was cleared for occupation by engineers provided some remedial work was done, she said.

"The engineer came out and said it held up really well and did what it needed to do ....it cost about $95,000 to repair it."

Kirk Roberts Consulting Engineers Ltd director Stephen Roberts, who worked on the building, said it had been built beyond the building code requirement and even exceeded the new tougher standards being recommended by the Institution of Professional Engineers since the February quake.

While the building had been damaged it had remained standing during three quakes that greatly exceeded the building code requirements, he said.

"The building has performed exceptionally well for the current building code," he said.

The environment in Christchurch since February was well beyond what any codes had predicted and if asked now he would not recommend proceeding with Crouch's building.

In fact it would be unwise to build anything in Christchurch until the risk of further severe aftershocks had passed and there was a better scientific understanding of the fault lines beneath the city, he said.

The building's architect, Marcus Stufkens, director of Stufkens and Chambers Architects, said he was not happy about the building's performance but it was too early to say whether there was any fault.

Crouch had asked for the building's performance to be peer-reviewed, he said.
Appendix E

Vibrocompaction Probe on Piling Rig

Intense vibration compacts gravels and sand, vastly improving performance under gravity and seismic loading. Improvement to depths of 20-30m can readily be achieved.
Appendix 16

‘Effect of Loading Rate on Anchorage Bond and Beam-Column Joints’
by Chung and Shah,
ACI Structural Journal
March-April 1989
Effect of Loading Rate on Anchorage Bond and Beam-Column Joints

by Lan Chung and Surendra P. Shah

Small-scale reinforced concrete specimens subjected to cyclic loading were loaded at rates varying from 0.0025 to 2.0 Hz to investigate the effects of several variables including loading rate, shear-span-to-depth ratio, and stirrup spacing. This study's primary objective was to investigate the rate effect of cyclic loading on bond in reinforced concrete structures. Several different parameters such as stiffness degradation, energy dissipation, failure mode, strain, and bond-slip distribution along the bar were examined to evidence the differences due to the rate of loading.

Keywords: anchorage, bond, strain, bond stress, failure modes, loading rate, cyclic loading, stiffness degradation, energy dissipation, and failure modes.

The current seismic codes for the design of reinforced concrete structures subjected to earthquake loading, such as the Universal Building Code, Structural Engineers Association of California code, and the ACI Building Code, are based on the experimental behavior of structural elements subjected to static cyclic loading. The rates at which these specimens were loaded were substantially lower than those corresponding to the rates which could be expected in earthquake, impact, and blast loadings.

Previous studies by several investigators have shown some influence of monotonically increased dynamic rates and some influence of cyclic loading rates conducted by the U.S. Army Waterways Experiment Station on the effect of explosives on buried reinforced concrete box structures as well as tests conducted by several investigators on reinforced concrete beams loaded at varying rates showed that concrete structures designed to fail in a ductile manner (flexural failure) under slow rates of loading may fail in a brittle manner (shear failure) at higher rates of loading. The primary objective of this study was to investigate the effect of rate of cyclic loading on the character of bond between reinforcement and concrete in reinforced concrete structures. Several different parameters including stiffness degradation, energy dissipation, failure mode, strain, and bond-slip distribution along the bar were examined to evaluate the effects of rate of loading.

RESEARCH SIGNIFICANCE

The current design of reinforced concrete structures is based on the experimental behavior of structural elements subjected to slow rates of loading. Concrete structures may be subjected to significantly faster rates of loading such as during earthquakes, impact, or blast. Research described in this paper indicates that the faster rates of loading may lead to fewer and wider cracks in the beam near the column face, which can cause brittle failure. The diagonal shear cracks emerge at earlier stages of loading for all test specimens. The specimens with stirrup spacing of 2/2 showed a marked influence of loading rate, whereas the specimens with stirrup spacing of 2/4 were not influenced by loading rate. Occurrence of shear cracks at small
d

Fig. 1 — Variation of failure mode with rate of loading (from Reference 2) (1 in. = 25.4 mm)

Fig. 2 — Variation of failure mode with rate of cyclic loading (from Reference 3) (1 in. = 25.4 mm)
The specimens were intended to simulate exterior beam-column joints of a moment-resisting reinforced concrete ductile frame. The primary variable was loading rate. The loading frequency of 0.0025 Hz was selected for all "slow-rate" specimens and frequencies of 0.1, 1.0, and 2.0 Hz were selected for the "fast-rate" specimens. The reinforcement was designed and detailed according to ACI 318 (Reference 1). The geometry of the specimen was selected to simulate the connection of a beam to a column. The length of the enlarged end of anchorage zone was chosen to ensure that the strength of the longitudinal reinforcement was developed by adequate anchorage of the straight portion of the bars. The beam and end block were cast monolithically and specimens were tested after two weeks of moist curing.

To measure the strains along the reinforcement with a minimum of interference with the bond mechanism, the reinforcing bars were half-cut and grooved. The reinforcing bar was made up of two parts, as shown in Fig. 3(a). The cut surface of each part was machined and the groove was milled for installation of strain gages. The technique involved milling two reinforcing bars down to a half-round and then machining a 0.1 in. (2.54 mm) wide × 0.05 in. (1.27 mm) deep longitudinal groove in each half-reinforcing bar to accommodate 12 strain gages [gage length = 0.051 in. (0.7874 mm)] and the associated wiring. The lead wire was secured to the reinforcing bar to insure that no tension force was exerted on the gage connection. All lead wires were waterproofed with a polyurethane coating and wax; the coating was then allowed to dry thoroughly before the wax was applied. After installing the gages and leads, the two halves were epoxied and spot-welded together so that the recombined bar had the appearance of a normal reinforcing bar. All gage leads were taken out through the hole at the end of the bar. Note that the details of all other specimens (Nos. 1 through 8) are similar to that shown in Fig. 3, except the reinforcing bars do not have grooves for strain gages (solid bars).

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<th>Table 1 — Specimen variables</th>
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Material properties
The properties of two types of steel used for beam longitudinal reinforcement (2 bars) and for shear reinforcement (D1 bars) are given in Table 2. The D1 bars used for shear reinforcement had special surface deformations to simulate deformed bars used in practice. Several investigators have shown that these specially knurled small-diameter bars adequately simulate the bond behavior of the full-sized bars.10

Concrete mixture proportions are also given in Table 2. Three 3 x 6 in. cylinders were cast and cured simultaneously with each specimen and subjected to uniaxial compressive loading on the same day the specimen was tested. The average modulus of elasticity (mean at 0.45 8) was 2900 ksi (20 MPa) and average compressive strength was 4000 psi (27.58 MPa).

Test specimens
The test specimens reinforced with longitudinal bars inside which strain gages were embedded are shown in Fig. 3(a) (Specimens 9 and 10) and 3(b) (Specimen 11). The load versus shear strain is plotted in Fig. 3(b) (Specimen 11). The deflection of the beam was measured using LVDTs (Fig. 4). The analog signals from the various measuring devices were conditioned, amplified, and digitized using an analog/digital converter and then recorded in a computer-based data acquisition system. The load versus load-point deflection was also recorded on an X-Y plotter.
specimens, diagonal shear cracks emerge at an earlier stage of loading than for the corresponding slow-rate specimens. The slow-rate specimens generally had a larger number of widely distributed cracks as compared to those for the fast-rate specimens.

**Load-deflection response**

Plots of applied load versus load-point deflection (P-u curves) are shown in Fig. 9(a) and (b) for Specimens 2 and 10 in Fig. 9(e) and (d) for Specimens 11 and 12. The ultimate loads of fast-rate specimens were higher (approximately 15 to 25 percent) than those of the slow-rate specimens. It can be seen from Fig. 9(e) and (d) that the fast-rate specimen (No. 12) that was tested at 2.0 Hz lost its strength during the fourth positive cycle due to the fracture of the reinforcing bar at the ductility ratio of 4.6 (A = 0.38 in. (9.65 mm)), while the slow-rate specimen failed during the fifth cycle at a much larger capacity (ductility ratio of 11.5, A = 0.95 in. (24.13 mm)).

For comparison, Fig. 10 plots the ratios of peak load, average stiffness, and energy dissipation capacity of fast-rate specimens (No. 2, 4, and 12) and those of the corresponding slow-rate specimens (No. 1, 3, and 11) at various displacement ductilities for specimens with stirrup spacing equal to d/2. From these figures, it is evident that for specimens with stirrup spacing equal to d/2, a fast rate of loading was more detrimental than a slow rate of loading. The greater damage due to higher rates of loading appears to depend on shear-span-to-depth ratio (compare Specimens 2 and 4) and rate of loading (Specimen 12). The strength of these fast-rate specimens was reduced to zero as a result of fracture of the reinforcing bar at the beam-column interface. In contrast, the fast-rate specimens 6, 8, and 10, which had a relatively larger amount of shear reinforcement (stirrup spacing = d/4), maintained load-carrying capacities similar to those of corresponding slow-rate specimens (Specimens 5, 7, and 9). This confirms that the requirements of transverse reinforcement in the ACI Building Code for seismic design (maximum spacing of transverse steel = d/4, ACI 318-93, Section A3.3.2) are adequate at higher rates of loading.

**EFFECT OF LOADING RATE**

Crack patterns

The effect of loading rate on mode of failure is illustrated in Fig. 8, which shows crack patterns for three sets of specimens. The specimens in each set had the same shear-span-to-depth ratio and identical stirrup spacing and were subjected to the same loading history. For each pair of specimens shown, the one on the right was subjected to a faster rate of loading. From such comparisons, it was observed that for all fast-rate specimens, diagonal shear cracks emerge at an earlier stage of loading than for the corresponding slow-rate specimens. The slow-rate specimens generally had a larger number of widely distributed cracks as compared to those for the fast-rate specimens.

**Testing procedure**

The loading schedules imposed at different rates on the specimens are shown in Fig. 5. Ductility ratio is defined here as the ratio of peak load to peak load at ultimate strain (the slope of the line which connects the original zero point and the peak load point of the first positive half cycle at each loading stage in the load-deflection curves), and energy dissipated (the area under the load-deflection hysteresis loop) during cyclic loading for model specimens. The specimens were then subjected to large-scale reversed loading. Further details of the experimental investigation as well as results of each specimen are given in Reference 8.

**Comparison with prototype**

To evaluate the validity of testing small-scale models, the results of the slow-rate Specimen 5 were compared with Specimens 66-32-BY-40 tested by Brown and Jirsa and Ismail and Jirsa. The dimensions and material properties of their specimens are shown in Fig. 6. From this figure it can be seen that (1) the dimensions of the prototype were four times that of the model; (2) the ratio of the prototype to model specimen sectional areas of longitudinal reinforcement is approximately the square of the length-scale factor; (3) the applied axial loads to enlarged block were 1 ksi (6895 kPa) for both prototype and model; and (4) concrete compressive strength was approximately 4000 and 3500 psi (27.58 and 24.54 MPa) for model and prototype, respectively. Both the model and the prototype specimens were subjected to the same loading history, as shown in Fig. 7(b).

To establish the reliability of small-scale modelling techniques, the ratio of peak load, average stiffness (the slope of the line which connects the original zero point and the peak load point of the first positive half cycle at each loading stage in the load-deflection curve), and energy dissipated (the area under the load-deflection hysteresis loop) during cyclic loading for model specimens were compared with those of the prototype. The ratios are plotted against the ductility ratio. For comparison purposes, the values obtained from the prototype were normalized according to the following requirements. Note that for each ductility ratio, the normalized prototype values were taken as unity. The overall agreement is good—the maximum difference in the reported values between prototype and models is less than 10 percent.
Strain distribution diagram

Data from the strain gages were used to calculate the strain distribution along the length of the longitudinal reinforcement bar. Strain distributions at the positive and negative peak loading from the first to the sixth cycles for Specimens 9 and 10, and the first two cycles for Specimens 11 and 12 are shown in Fig. 12(a) and (b) and 13(a) and (b), respectively. The measured strains are total strains referred to nominal zero value, which corresponds to the initial condition just prior to the first loading of the specimens. The influence due to shrinkage or thermal effects on the gage was neglected. The notations A and B in the figures refer to positive and negative peak at each cycle, respectively. Fig. 12(a) and (b) show that the strain in Specimen 10 are higher and have a higher gradient than those in Specimen 9. In the slow-rate specimens, shows of the strain-distribution curves were flatter and strains eventually reached the loaded end. For example, for Specimen 9, strains were transferred to the unloaded end at the twenty-second cycle and 0.0004 to 0.0005 in. (0.01016 to 0.01270 mm) end-slip were observed at this cycle from the dial gages mounted at the end of the bar. No end-slip was observed for the fast-rate specimen. Takeda\(^{10}\) has studied the rate effect on the bond-stress distribution in deformed bars during a pullout test; he too observed that the bond stresses have a steeper gradient at a faster rate of loading.

Strains distributed in the reinforcement located along the beam portion of Specimens 11 and 12 are shown in Fig. 13. The fast-rate specimen (No. 12) showed much higher strains at the beam-column interface than the slow-rate specimen (No. 11). Note that Specimen 12 failed earlier at the beam-column interface by rupturing of the steel. The more uniform strain distribution near the column face for the slow-rate specimen indicates a possible loss of bond near the column face for the slow-rate specimen. For slow-rate specimens, peak values of strains were also observed away from the critical face, indicating internal cracking and a wider distribution of cracking as compared to fast-rate specimens, for which the maximum value of strain was always at the column face.

Total elongations of the reinforcing bar were flatter and strains eventually reached the loaded end. For example, for Specimen 9, strains were transferred to the unloaded end at the twenty-second cycle and 0.0004 to 0.0005 in. (0.01016 to 0.01270 mm) end-slip were observed at this cycle from the dial gages mounted at the end of the bar. No end-slip was observed for the fast-rate specimen. Takeda\(^{10}\) has studied the rate effect on the bond-stress distribution in deformed bars during a pullout test; he too observed that the bond stresses have a steeper gradient at a faster rate of loading.

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Fewer, and consequently wider, cracks for fast-rate specimens may mean a reduced shear resistance due to aggregate interlock phenomenon. During earlier stages of loading, cracks were also longer for fast-rate specimens. The wider and longer cracks may explain the observed premature degradation of shear resistance for the fast-rate specimen.

CONCLUSIONS

Based on the experimental test results obtained in this study, the following conclusions can be drawn:

1. Some specimens subjected to faster rates of loading failed (due to the premature fracture of steel bar) at a lower ductility ratio compared to the corresponding slow-rate specimen. The observed differences are associated with the concentration of strains caused by improved bond strength at faster rates.

2. At faster rates of loading, fewer and wider cracks were observed near the column face, whereas more widely distributed cracks were observed in the beam parts of the beam-column and anchorage-bond specimens. The loss of aggregate interlock associated with large crack widths results in an earlier decrease in the shear strength at the fast-rate of loading compared to the slow-rate of loading.

3. The specimen with the stirrup spacing of d/2 showed a marked influence of the loading rate. For this set of specimens, the fast rate of loading resulted in a brittle mode of failure when compared with the slow rate of loading. In contrast, the specimen with stirrup spacing of d/4 was not influenced by loading rate.

4. The effect of loading rate described probably depends on the interaction of the dimensions of the specimen (d, d/d), bond mechanism between steel and concrete, and reinforcing parameters. Occurrence of ACI Structural Journal / March-April 1989
ACKNOWLEDGMENT

The research described in this paper was conducted under Grant AFOSR 84-0028 from the Air Force Office of Scientific Research. The authors appreciate the support.

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Shear Strength of Prestressed Concrete Beams with Unbonded Tendons

by K. Kordina, J. Hegger, and M. Teutsch

The results of shear tests on 10 precast concrete beams with unbonded tendons are described. The purpose of the tests was to elucidate the effect of the bond condition upon the shear-bearing behavior of beams. Evaluation of test results and comparison with previous studies show that the shear strength of prestressed beams with unbonded tendons can be determined correctly with a truss model. Conversely, a tied-arch model is found to achieve inadequate agreement with the experimentally determined shear strength. For beams with bonded longitudinal reinforcement, depending on the degree of web reinforcement, two failure modes—tension or flexural shearing failure and web-crushing failure—must be distinguished. For both failure modes, new regulations for pre stressing with bonded tendons can be used.

Although there have been numerous reports on the flexural strength behavior of prestressing without bond, only a few reports on the shear-carrying behavior have been published. The reason for this low level of research activity in this field is that in recent years prestressed unbonded tendons has been used primarily for slab structures with re call low shear stresses, although there are considerable economical and operational possibilities for beam-like structures. Therefore, fundamental investigations in the field on the shear-carrying behavior exist only for “punching,” and so far the shear-carrying behavior of beams has been investigated only by Lorenzen and Jena and Pazzelli for beams without web reinforcement.

For beams with bonded tendons after the first shear cracks have occurred, the shear-carrying system can be described appropriately by truss models, as proved by numerous investigations. According to the truss analogy, the shear force is carried by the compression struts, whose compressive force is balanced by the stirrup and tension chord forces (Fig. 1). The resulting horizontal component $\Delta z$ must be transferred by bond into the tensile reinforcement (Fig. 1).

For a beam with only unbonded reinforcement, the horizontal force $\Delta z$ is necessary for the equilibrium at the ACI Structural Journal / March-April 1989
Appendix 17

Building (Designation of Building Work Licensing Classes) Order 2010
Building (Designation of Building Work Licensing Classes) Order 2010

Anand Satyanand, Governor-General

Order in Council

At Wellington this 1st day of March 2010

Present:
His Excellency the Governor-General in Council

Pursuant to sections 285 and 291 of the Building Act 2004, His Excellency the Governor-General, acting on the advice and with the consent of the Executive Council, and on the recommendation of the Minister (as defined by section 7 of that Act) made, as required by section 403(2) of that Act, after he or she became satisfied that the chief executive (as so defined) has consulted in accordance with section 403(3) and (4) of that Act, makes the following order.

Contents

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Title</td>
</tr>
<tr>
<td>2</td>
<td>Commencement</td>
</tr>
<tr>
<td>3</td>
<td>Interpretation</td>
</tr>
</tbody>
</table>
Designation of building work licensing classes

4 Licensing classes designated and building work that LBPs are licensed to carry out or supervise

Automatic licensing of certain professions

5 Specified architects, engineers, plumbers, and gasfitters treated as if licensed in specified licensing classes

Transitional provisions and revocation

6 Licensing in discontinued licensing class converts to licensing in new licensing class

7 Revocation

Schedule

Categorisation of buildings

Order

1 Title
This order is the Building (Designation of Building Work Licensing Classes) Order 2010.

2 Commencement
This order comes into force on 1 April 2010.

3 Interpretation
(1) In this order, unless the context otherwise requires,—
  Act means the Building Act 2004
  ancillary building means a building classified under clause A1 of the building code as under the category of ancillary (for example, a bridge, derrick, fence, free-standing outdoor fireplace, jetty, mast, path, platform, pylon, retaining wall, tank, tunnel, or dam)
  area of practice 1, area of practice 2, and area of practice 3, in relation to licensing, or an application to be licensed, in a licensing class, have the same meanings as in the rules
  category 1 building, category 2 building, and category 3 building have the meanings given to them by the Schedule
  LBP means a licensed building practitioner
**NZS 3604:1999** means the New Zealand standard on *Timber framed buildings* as amended by the following:
(a) NZS 3604:1999A1: Amendment 1; and
(b) NZS 3604:1999A2: Amendment 2

**outbuilding** means a building classified under clause A1 of the building code as under the category of outbuildings (for example, a carport, farm building, garage, greenhouse, machinery room, private swimming pool, public toilet, or shed)

**standard** has the meaning given to it by section 2 of the Standards Act 1988.

(2) Terms or expressions used and not defined in this order but defined in the Act have, in this order, the same meanings as they have in the Act.

### Designation of building work licensing classes

4 **Licensing classes designated and building work that LBPs are licensed to carry out or supervise**

This order designates the licensing classes specified in rows 1 to 9 of the following table, and an LBP who is licensed in a class specified in one of those rows is licensed to carry out or supervise building work of the type specified in that row:

<table>
<thead>
<tr>
<th>Row</th>
<th>Licensing class</th>
<th>Type of building work</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>General licensing classes</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Design</td>
<td>Design work for any building that is—</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(a) a category 1 building; or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(b) a category 2 building; or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(c) a category 3 building</td>
</tr>
<tr>
<td>2</td>
<td>Site</td>
<td>Co-ordination or oversight of some or all of the construction or alteration of any building that is—</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(a) a category 1 building; or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(b) a category 2 building; or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(c) a category 3 building</td>
</tr>
<tr>
<td>Row</td>
<td>Licensing class</td>
<td>Type of building work</td>
</tr>
<tr>
<td>-----</td>
<td>--------------------------</td>
<td>---------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td></td>
<td><strong>Trade licensing classes</strong></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Carpentry</td>
<td>Carpentry for any building that is—</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(a) a category 1 building; or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(b) a category 2 building; or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(c) a category 3 building</td>
</tr>
<tr>
<td>4</td>
<td>Roofing</td>
<td>Installation of roofs, or roofing materials, for any building that is—</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(a) a category 1 building; or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(b) a category 2 building; or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(c) a category 3 building</td>
</tr>
<tr>
<td>5</td>
<td>External plastering</td>
<td>Application of external solid plaster, or proprietary plaster systems, to any building</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(a) a category 1 building; or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(b) a category 2 building; or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(c) a category 3 building</td>
</tr>
<tr>
<td>6</td>
<td>Bricklaying and blocklaying</td>
<td>Laying or erection of bricks or blocks for any building that is—</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(a) a category 1 building; or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(b) a category 2 building; or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(c) a category 3 building</td>
</tr>
<tr>
<td>7</td>
<td>Foundations</td>
<td>Construction or alteration of some or all of the foundations in or for any building</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(a) a category 1 building; or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(b) a category 2 building</td>
</tr>
<tr>
<td></td>
<td><strong>Specialist licensing classes</strong></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Concrete structure</td>
<td>Co-ordination or oversight of some or all of the construction or alteration of 1 or more</td>
</tr>
<tr>
<td></td>
<td></td>
<td>concrete structures in or for any building that is—</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(a) a category 1 building; or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(b) a category 2 building; or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(c) a category 3 building</td>
</tr>
<tr>
<td>9</td>
<td>Steel structure</td>
<td>Co-ordination or oversight of some or all of the construction or alteration of 1 or more</td>
</tr>
<tr>
<td></td>
<td></td>
<td>steel structures in or for any building that is—</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(a) a category 1 building; or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(b) a category 2 building; or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(c) a category 3 building</td>
</tr>
</tbody>
</table>
Automatic licensing of certain professions

5 Specified architects, engineers, plumbers, and gasfitters treated as if licensed in specified licensing classes

(1) People specified in column 1 of a row of the following table are, under section 291(2) of the Act, treated as if they were licensed in the building work licensing class or classes specified (and as a result of an assessment in respect of the area of practice, if any, specified after that class or those classes) in column 2 of that row unless their registration, licence, or other recognition under the Act specified in that row is suspended or cancelled:

<table>
<thead>
<tr>
<th>Column 1</th>
<th>Column 2 Building work licensing class(es)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Row</td>
<td>Profession</td>
</tr>
<tr>
<td>1</td>
<td>Registered architects under the Registered Architects Act 2005</td>
</tr>
<tr>
<td>2</td>
<td>Chartered professional engineers under the Chartered Professional Engineers of New Zealand Act 2002</td>
</tr>
<tr>
<td>3</td>
<td>People registered or deemed to be registered under the Plumbers, Gasfitters, and Drainlayers Act 2006 in respect of a class of registration that, with or without modification, replaces or corresponds to craftsman plumber or registered plumber</td>
</tr>
<tr>
<td>4</td>
<td>People registered or deemed to be registered under the Plumbers, Gasfitters, and Drainlayers Act 2006 in respect of a class of registration that, with or without modification, replaces or corresponds to craftsman gasfitter or registered gasfitter</td>
</tr>
</tbody>
</table>

(2) Column 1 of row 3 of the table in subclause (1) must until the commencement of sections 41 and 173 of the Plumbers, Gasfitters, and Drainlayers Act 2006 be treated as if it referred only to people who are craftsman plumbers or registered plumbers under the Plumbers, Gasfitters, and Drainlayers Act 1976.
Column 1 of row 4 of the table in subclause (1) must until the commencement of sections 41 and 173 of the Plumbers, Gasfitters, and Drainlayers Act 2006 be treated as if it referred only to people who are craftsman gasfitters or registered gasfitters under the Plumbers, Gasfitters, and Drainlayers Act 1976.

**Transitional provisions and revocation**

6 Licensing in discontinued licensing class converts to licensing in new licensing class

(1) Licensing that is in a class specified in column 1 of a row of the following table and that is current at the close of 31 March 2010 continues after 31 March 2010 until cancelled or suspended but after 31 March 2010 is treated as if it were licensing in the class specified in column 2 of that row and arising from an assessment in respect of the area of practice specified in column 3 of that row:

<table>
<thead>
<tr>
<th>Row</th>
<th>Discontinued licensing class</th>
<th>New licensing class</th>
<th>Area of practice</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Design—class 1</td>
<td>Design</td>
<td>Area of practice 1</td>
</tr>
<tr>
<td>2</td>
<td>Design—class 2</td>
<td>Design</td>
<td>Area of practice 2</td>
</tr>
<tr>
<td>3</td>
<td>Design—class 3</td>
<td>Design</td>
<td>Area of practice 3</td>
</tr>
<tr>
<td>4</td>
<td>Site—class 1</td>
<td>Site</td>
<td>Area of practice 1</td>
</tr>
<tr>
<td>5</td>
<td>Site—class 2</td>
<td>Site</td>
<td>Area of practice 2</td>
</tr>
<tr>
<td>6</td>
<td>Site—class 3</td>
<td>Site</td>
<td>Area of practice 3</td>
</tr>
</tbody>
</table>

An application for licensing in a class specified in column 1 of a row of the table in subclause (1) and that at the close of 31 March 2010 has not been withdrawn or finally determined continues after 31 March 2010 as if this order had not been made except that any licensing resulting from the granting of the application is treated as if it were licensing in the class specified in column 2 of that row and arising from an assessment in respect of the area of practice specified in column 3 of that row.
7 Revocation
The Building (Designation of Building Work Licence Classes) Order 2007 (SR 2007/126) is revoked.

______________________________

Schedule  
cl 3(1)

Categorisation of buildings

Part 1
Category 1 buildings
A category 1 building is a building—
(1) that is neither an ancillary building nor an outbuilding; and
(2) whose building envelope (whether the building is a new structure or an existing building) has a total risk score (calculated under Part 4) that does not exceed 12 for any external elevation; and
(3) whose use (determined, whether the building is a new structure or an existing building, in accordance with regulation 6 and Schedule 2 of the Building (Specified Systems, Change the Use, and Earthquake-prone Buildings) Regulations 2005) is SH (sleeping single home).

Part 2
Category 2 buildings
A category 2 building is a building that is—
(1) neither an ancillary building nor an outbuilding; and
(2) neither a category 1 building nor a category 3 building.

Part 3
Category 3 buildings
A category 3 building is a building that is neither an ancillary building nor an outbuilding, and that is not a category 1 building, but is a building—
(1) whose use (determined, whether the building is a new structure or an existing building, in accordance with regulation 6 and Schedule 2 of the Building (Specified Systems, Change the
Part 3—continued

 Use, and Earthquake-prone Buildings) Regulations 2005) is not SH (sleeping single home); and

(2) whose building height is or exceeds 10 metres where—

**building height** means the vertical distance between the upper surfaces of the floors of the building’s lowest and highest storeys

**floor** does not include a mezzanine floor or a rooftop area

**storey** means a portion of the building between the upper surface of a floor (floor A) in the building and—

(a) the upper surface of the next above floor in the building;

or

(b) if there is no next above floor in the building, the lower surface of the ceiling or roof above floor A.

Part 4

Total risk score

(1) A total risk score for an external elevation on the building envelope of a building is calculated by using the calculation table in clause (2) and completing the following steps:

(a) for each specified risk factor, use the definitions of risk table in clause (3) to decide whether the risk severity is low, medium, high, or very high, and enter the applicable score; and

(b) copy the applicable score for each risk factor to the column whose heading is “Subtotals for each risk factor”; and

(c) add all the scores copied into that column to arrive at the total risk score for the external elevation.
Part 4—continued

(2) The calculation table is as follows:

<table>
<thead>
<tr>
<th>Risk factor</th>
<th>Low</th>
<th>Medium</th>
<th>High</th>
<th>Very high</th>
<th>Subtotals for each risk factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind zone</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Number of storeys</td>
<td>0</td>
<td>1</td>
<td>2</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>Roof/elevation intersection design</td>
<td>0</td>
<td>1</td>
<td>3</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>Eaves width</td>
<td>0</td>
<td>1</td>
<td>2</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>Envelope complexity</td>
<td>0</td>
<td>1</td>
<td>3</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>Deck design</td>
<td>0</td>
<td>2</td>
<td>4</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td><strong>Total risk score:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(3) The definitions of risk table is as follows:

<table>
<thead>
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<td>Roof-to-elevation intersection</td>
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<td>fully protected (for example, hip</td>
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<td>and gable roof with eaves)</td>
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### Part 4—continued

#### Definitions of risk table

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<th>Level</th>
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<tr>
<td>Medium risk</td>
<td>Roof-to-elevation intersection partly exposed (for example, hip and gable roof with no eaves)</td>
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<tr>
<td>High risk</td>
<td>Roof-to-elevation intersection fully exposed (for example, parapets, enclosed balustrades, or eaves at greater than 90° to vertical with soffit lining)</td>
</tr>
<tr>
<td>Very high risk</td>
<td>Roof elements finishing within the boundaries formed by the external elevations (for example, lower ends of aprons, chimneys, dormers, etc)</td>
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</table>

<table>
<thead>
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<th>Eaves width&lt;sup&gt;(1)(2)&lt;/sup&gt;</th>
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<td>Greater than 600 mm for single storey</td>
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<tr>
<td>Medium risk</td>
<td>451–600 mm for single storey, or greater than 600 mm for 2 storey</td>
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<tr>
<td>High risk</td>
<td>101–450 mm for single storey, or 451–600 mm for 2 storey, or greater than 600 mm above 2 storey</td>
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<tr>
<td>Very high risk</td>
<td>0–100 mm for single storey, or 0–450 mm for 2 storey, or 600 mm or less above 2 storey</td>
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<th>Envelope complexity</th>
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<td>Simple rectangular, L, T, or boomerang shape, with single cladding type</td>
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<tr>
<td>Medium risk</td>
<td>Moderately complex, angular, or curved shapes (for example, Y or arrowhead) with no more than 2 cladding types</td>
</tr>
<tr>
<td>High risk</td>
<td>Complex, angular, or curved shapes (for example, Y or arrowhead) with multiple cladding types</td>
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</tbody>
</table>
Part 4—continued

Definitions of risk table

**Very high risk**
As for high risk, but with junctions not covered in the rows of this table relating to roof/elevation intersection design and deck design (for example, box windows, pergolas, or multi-storey re-entrant shapes)

**Deck design**¹

| Low risk | None, timber slat deck or porch at ground-floor level |
| Medium risk | Fully covered in plan by roof, or timber slat deck attached at first- or second-floor level |
| High risk | Enclosed deck exposed in plan or cantilevered at first-floor level |
| Very high risk | Enclosed deck exposed in plan or cantilevered at second-floor level or above |

Notes

(1) Eaves width measured horizontally from external face of wall cladding to outer edge of overhang, including gutters and fascias.
(2) Balustrades and parapets count as 0 mm eaves.
(3) **Deck**—
   (a) means an open platform (which may be known as a balcony)—
      (i) projecting from an exterior wall of a building; and
      (ii) supported by framing; and
      (iii) that may be over enclosed internal spaces, or may be open underneath; and
   (b) includes an enclosed deck (that is, a deck (which may be known as a balcony)—
      (i) over an interior or exterior space; and
      (ii) that has an impermeable upper surface; and
      (iii) that is closed on the underside).

Rebecca Kitteridge,
Clerk of the Executive Council.
Explanatory note

This note is not part of the order; but is intended to indicate its general effect.

This order, which comes into force on 1 April 2010, is made under the Building Act 2004 (the Act). The order—

• designates 9 building work licensing classes for licensed building practitioners under the Act; and

• specifies the types of building work that licensed building practitioners who are licensed in those classes are licensed to carry out or supervise; and

• specifies people who, under section 291(2) of the Act, are treated as if they are licensed in a specified class.

Rules relating to licensed building practitioners are, under sections 353 to 362 of the Act, prepared by the chief executive of the Department of Building and Housing, and then approved by the Building Practitioners Board and the Minister for Building and Construction. Rules of that kind contain the following minimum standards:

• minimum standards of competence (including standards relating to knowledge and skills) that must be met for each licensing class; and

• minimum standards for demonstrating current competence for each licensing class that must be met for continued licensing, and for the frequency at which assessments of current competence must be carried out.

Section 84 of the Act requires all restricted building work to be carried out or supervised by a licensed building practitioner who is licensed to carry out or supervise the work. However, section 84 and other provisions of the Act that relate to restricted building work are yet to come into force. Before their commencement, there is no kind of building work—

• that may be lawfully carried out or supervised by a licensed building practitioner only; or

• that cannot lawfully be carried out or supervised by a licensed building practitioner who is licensed to carry out or supervise building work of a certain type only.

Licensing does, however, still have some consequences under the Act before the commencement of the restricted building work provisions. For example, the register of building practitioners established
and maintained under section 298 of the Act enables members of the public to choose a suitable building practitioner from a list of licensed building practitioners. Licensed building practitioners may also, under subpart 2 of Part 4 of the Act, be the subject of complaints to, and disciplinary penalties imposed by, the Building Practitioners Board. In addition, a person commits an offence under section 314 of the Act if the person—

- holds himself or herself out as a person who is licensed to carry out or supervise building work, or building work of a certain type, while not being so licensed; or
- is a licensed building practitioner and fails on a request for the purpose to produce for inspection by the requester material that is, or is a copy of, evidence of being licensed, as required by section 289 of the Act; or
- is a person applying to become licensed, or a licensed building practitioner, and fails to give written notice of a change in circumstances, in accordance with section 302 of the Act.

This order revokes and replaces the Building (Designation of Building Work Licence Classes) Order 2007. The main changes made by this order are as follows:

- a new foundations licensing class is established relating to construction or alteration of foundations:
- the former building services licensing class is discontinued (no licensing in that class has occurred, in part because no rules prescribing minimum standards for that class have been made):
- the former 3 design licensing classes are replaced with a single design licensing class, the former 3 site licensing classes are replaced with a single site licensing class, and relevant existing licensing is converted accordingly:
- Chartered professional engineers under the Chartered Professional Engineers of New Zealand Act 2002 are automatically licensed not only in the design licensing class, but also in the site licensing class:
- registered plumbers and registered gasfitters are automatically licensed in the roofing, external plastering, and bricklaying and blocklaying licensing classes:
• the definitions of the 3 categories of buildings are simplified.

Issued under the authority of the Acts and Regulations Publication Act 1989.
Date of notification in Gazette: 4 March 2010.
This order is administered by the Department of Building and Housing.
Appendix 18

Figure C5.A5 of NZS 3101:Part2:2006
NZS 3101:Part 2:2006

s    spacing of stirrups (mm)

z    the internal lever arm corresponding to the maximum bending moment in the element under consideration (mm) in a member with constant depth

An example of the standard method, i.e.: $\theta = 45^\circ$, will be used for the shear analysis.

![Diagram of strut and tie model](image)

**Figure C5.A5 – Strut and tie model**

C5.A4.2.1 Standard method

The design shear resistance of a section of a beam with shear reinforcement and containing steel fibres is given by the equation:

$$V_{d,3} = V_b + V_{in} + V_{ad}$$

(Eq. C5A–8)

with

$$V_b = k_d k_3 (0.07 + 10 \rho_w \sqrt{f_{c} b_n d}$$

(Eq. C5A–9)

where

$V_b$ shall not be more than $0.2 \sqrt{f_{c} b_n d}$ nor need be less than $0.08 \sqrt{f_{c} b_n d}$, and $k_d$ and $k_3$ are given by 9.3.9.3.4.

where

$$\rho_w = \frac{A_{sa}}{b_w d}$$

(Eq. C5A-10)

where

$A_{sa}$ is the area of tension reinforcement bars extending equal to or greater than "d + anchorage length" beyond the section considered, Figure C5.A6, mm$^2$

$b_w$ is the minimum width of the section over the effective depth $d$ (mm).
Appendix 19

NZCIC Membership List
<table>
<thead>
<tr>
<th>Name</th>
<th>Role</th>
<th>Organization</th>
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<th>Phone</th>
<th>Mobile</th>
<th>Email</th>
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</tr>
</thead>
<tbody>
<tr>
<td>Pieter Burghout</td>
<td>CHAIRMAN</td>
<td>BRANZ Ltd</td>
<td>Private Bag 50908, PORIRUA 5240</td>
<td>04 238 1293</td>
<td>027 277 2469</td>
<td><a href="mailto:pieter.burghout@branz.co.nz">pieter.burghout@branz.co.nz</a></td>
<td><a href="http://www.branz.co.nz">www.branz.co.nz</a></td>
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<tr>
<td>Beverley McRae</td>
<td>DEPUTY CHAIR</td>
<td>BRANZ</td>
<td></td>
<td>09 623 6082</td>
<td>021 378 838</td>
<td><a href="mailto:bmcrae@nzia.co.nz">bmcrae@nzia.co.nz</a></td>
<td><a href="http://www.nzia.co.nz">www.nzia.co.nz</a></td>
</tr>
<tr>
<td>Derek Baxter</td>
<td>TREASURER</td>
<td>SNZ</td>
<td>Private Bag 2439, WELLINGTON 6140</td>
<td>04 498 3954</td>
<td>021 340 535</td>
<td><a href="mailto:Derek.Baxter@standards.co.nz">Derek.Baxter@standards.co.nz</a></td>
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<tr>
<td>Kieran Shaw</td>
<td></td>
<td>ACENZ</td>
<td>PO Box 10247, The Terrace, WELLINGTON 6143</td>
<td>04 472 1202</td>
<td>027 457 4303</td>
<td><a href="mailto:ksceo@acenz.org.nz">ksceo@acenz.org.nz</a></td>
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</tr>
<tr>
<td>Astrid Andersen</td>
<td>Managing Director</td>
<td>ADNZ</td>
<td>PO Box 39147, Harewood, CHRISTCHURCH 8545</td>
<td>03 358 0112</td>
<td>027 494 4429</td>
<td><a href="mailto:astrid@adnz.org.nz">astrid@adnz.org.nz</a></td>
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<tr>
<td>Ruma Karaitiana</td>
<td>Chief Executive</td>
<td>BCITO</td>
<td>PO Box 2615, WELLINGTON 6140</td>
<td>04 381 6439</td>
<td>027 466 5635</td>
<td><a href="mailto:ruma.karaitiana@bcito.org.nz">ruma.karaitiana@bcito.org.nz</a></td>
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<tr>
<td>Bruce Kohn</td>
<td>Chief Executive</td>
<td>BIFNZ</td>
<td>10 Treasure Grove, Hataitai, WELLINGTON 6021</td>
<td>04 386 2793</td>
<td>027 247 7748</td>
<td><a href="mailto:bruce.kohn@xtra.co.nz">bruce.kohn@xtra.co.nz</a></td>
<td><a href="http://www.bifnz.org.nz">www.bifnz.org.nz</a></td>
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<td>Nicholas (Nick)</td>
<td>Chief Executive</td>
<td>BOINZ</td>
<td>PO Box 11424, Manners Street, WELLINGTON 6142</td>
<td>04 473 6006</td>
<td>027 431 4607</td>
<td><a href="mailto:nickhill@boinz.org.nz">nickhill@boinz.org.nz</a></td>
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<td>PO Box 13405, TAURANGA 3141</td>
<td>07 927 7720</td>
<td>021 906 487</td>
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<td>DBH</td>
<td>PO Box 10729</td>
<td>04 494 0260</td>
<td>027 444 5559</td>
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<td>DINZ</td>
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<td>04 387 3678</td>
<td>027 591 0034</td>
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<td>Private Bag 302372</td>
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<td>Wolfgang Scholz</td>
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<td>HERA</td>
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<td>09 262 4848</td>
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<tr>
<td>Nicki Crauford</td>
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<td>IPENZ</td>
<td>PO Box 12241</td>
<td>04 474 8932</td>
<td>021 452 677</td>
<td>D <a href="mailto:CE@ipenz.org.nz">CE@ipenz.org.nz</a></td>
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<tr>
<td>Carl Davies</td>
<td>National Assn of Steel-Framed Housing Inc</td>
<td>NASH</td>
<td>PO Box 76134</td>
<td>09 262 1625</td>
<td>021 963 895</td>
<td><a href="mailto:gm@nashnz.org.nz">gm@nashnz.org.nz</a></td>
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<tr>
<td>Jeremy Sole</td>
<td>Chief Executive</td>
<td>NZCF</td>
<td>PO Box 12013</td>
<td>04 496-3273</td>
<td>021 777 646</td>
<td><a href="mailto:Jeremy@nzcontractors.co.nz">Jeremy@nzcontractors.co.nz</a></td>
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<tr>
<td>Alex Cutler</td>
<td>Chief Executive</td>
<td>NZGBC</td>
<td>PO Box 5286</td>
<td>09 379 3996</td>
<td>021 343 531</td>
<td><a href="mailto:alex.cutler@nzgbc.org.nz">alex.cutler@nzgbc.org.nz</a></td>
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<tr>
<td>Philip O'Sullivan</td>
<td>President</td>
<td>NZIBS</td>
<td>PO Box 1283</td>
<td>03 455 1499 or 0800 11 34 00</td>
<td>021 347 598 (P O'Sullivan)</td>
<td><a href="mailto:phil@prendos.co.nz">phil@prendos.co.nz</a></td>
<td><a href="http://www.buildingsurveyor.co.nz">www.buildingsurveyor.co.nz</a></td>
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<tr>
<td>Alt. Neville Scott</td>
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<td><a href="mailto:Neville.Scott@xtea.co.nz">Neville.Scott@xtea.co.nz</a></td>
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<tr>
<td>Andrew Gray</td>
<td>NZ Institute of Landscape Architects</td>
<td>NZILA</td>
<td>c/- PO Box 50218</td>
<td>04 237 1509</td>
<td></td>
<td><a href="mailto:agray@pcc.govt.nz">agray@pcc.govt.nz</a></td>
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<tr>
<td>Rick Mason</td>
<td>Chief Executive</td>
<td>NZ Institute of Building Inc</td>
<td>North Harbour</td>
<td>09 448 1911</td>
<td>021 845 107</td>
<td><a href="mailto:rick@nziob.org.nz">rick@nziob.org.nz</a></td>
<td><a href="http://www.nziob.org.nz">www.nziob.org.nz</a></td>
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<tr>
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<td></td>
<td>AUCKLAND 1330</td>
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</tr>
<tr>
<td>John Granville</td>
<td>Executive Director</td>
<td>NZ Institute of Quantity Surveyors</td>
<td>The Terrace</td>
<td>04 473 5521</td>
<td></td>
<td><a href="mailto:johngranville@nziqs.co.nz">johngranville@nziqs.co.nz</a></td>
<td>w.nziqs.co.nz</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>WELLINGTON 6143</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Connal Townsend</td>
<td></td>
<td>Property Council of New Zealand</td>
<td>Shortland Street</td>
<td>09 373 3086</td>
<td>021 781 482</td>
<td><a href="mailto:connal@propertynz.co.nz">connal@propertynz.co.nz</a></td>
<td><a href="http://www.propertynz.co.nz">www.propertynz.co.nz</a></td>
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<td>David Clark</td>
<td>Chief Executive</td>
<td>Property Institute of NZ</td>
<td>Manners Street</td>
<td>04 384 7094</td>
<td>027 443 8759</td>
<td><a href="mailto:david@property.org.nz">david@property.org.nz</a></td>
<td><a href="http://www.property.org.nz">www.property.org.nz</a></td>
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<td>Pamela Bell</td>
<td>Chief Executive</td>
<td>PrefabNZ Incorporated</td>
<td>Courtenay Place</td>
<td>021 972 635</td>
<td></td>
<td><a href="mailto:pam@prefabnz.com">pam@prefabnz.com</a></td>
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<td>Warwick Quinn</td>
<td>Chief Executive</td>
<td>Registered Master Builders Federation</td>
<td>Marion Square</td>
<td>04 494 8339</td>
<td>027 497 7787</td>
<td><a href="mailto:gmenhennet@sitesafe.org.nz">gmenhennet@sitesafe.org.nz</a></td>
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<td>Debbie Chin</td>
<td>Chief Executive</td>
<td>Standards New Zealand</td>
<td>Marion Square</td>
<td>04 801 2013</td>
<td>027 430 3629</td>
<td><a href="mailto:fiona@masterplumbers.org.nz">fiona@masterplumbers.org.nz</a></td>
<td><a href="http://www.masterplumbers.org.nz">www.masterplumbers.org.nz</a></td>
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June 2012