Report to the Canterbury Eathquakes Royal Commission. July 2012.

Discussion paper: training and education of engineers and organisation of the engineering profession.

Mr. Justice Cooper, Sir Ron Carter and Professor Richard Fenwick

Preliminary Remarks;

Gentlemen,

Stand back for a moment, and do not get lost in the minutiae of matters raised at the Commission hearings, but rather consider the salient, unassailable facts that must be addressed, before any logical report on the inadequacies of the Code approach, to the present structural design methodology, currently adopted in the design of reinforced concrete buildings. To adopt a positive approach to the amelioration of the lack of standards, in the structural section of engineering, particularly the present approach to earthquake design, it is first necessary to confront the reality of those areas which require immediate attention. After a long productive, successful career in engineering specialising in structural design, I am appalled by the alarming lack of competence, demonstrated by some people, masquerading as structural design engineers.

Frankly, I approach this submission with some reluctance, as the structural design theory emanates from universities, one in particular, and has been inculcated into students, thus into design offices, structural codes are into accepted building design practice. These ideas and concepts, now firmly implanted, will take intellectual dynamite to shift. Students are taught what to think not how to think. Thus, I would regard my submission as a fruitless exercise, but for the fact that Richard Fenwick is a member of the Commission, as I consider he represents an exception to the University genre, as he has for years, attempted to protect scientific rigour and academic honesty. This may not be construed, to mean that I concur with everything he has to say, but I share many of his views and respect his integrity. Structural engineering, in theory, design and site execution has always been very important to me, and I am saddened, by its present state, where there are now instances, where the public is at risk. (The Vector Arena and Stadium Southland, to name but 2 buildings.) Because of this, and because I have had a long career, in which to witness the descent into this maelstrom of stupidity, I proffer my overview of the pathway that has led us to this parlous condition. I do this, despite my experience that has found that most people in authority sometimes have a problem with inconvenient truth, and that successful careers often depend on singing from the acceptably correct song-sheet.

1. Legislation: CPEng. Act. 2002

This was an ill-conceived Act, as it was but a pale copy of the English methodology of registering engineers as C.Eng. (Royal Chartered Engineer). Before being granted C.Eng., one must first, be qualified by examination, conducted by one of the various Institutions of Engineers, i.e. Inst. of Mechanical Engineers, Inst. of Structural Engineers, Inst. of Civil Engineers, Inst. of Electrical Engineers, and so on through their various disciplines. Only then, are they proposed and finally certified as Chartered Engineer. Indeed, I was instructed by my Institution to designate myself as a Chartered Structural Engineer. By this method of designation, there can be no possible confusion of the expertise of any practising engineer. With reference to the **Discussion Paper, Cl. 5.2** in which it is stated, '*The CPEng register is available for public inspection. It records the name of the engineer and the date at which he* *or she was last registered or re-registered as a CPEng**. It has been suggested that the identification of CPEng's scope of practice would be useful to*** **the public'** My emphasis; who was the sensible person who suggested this basic, logical designation of engineers? Presently, the system, not only confuses the public but even the Cabinet Minister, Mr Williamson, who is supposedly in charge of these matters. I wrote in an endeavour to clarify things for him:

'*What has prompted me to e-mail you this letter was your unbelievable performance on Mark Sainsbury's Close-Up program, with John Scarry. At best, it appeared to be a prime example of obfuscation, by you, and at worst, insincerity on an impressive scale, even for a politician. You tried the normal ad hominem argument with John Scarry, instead of debating his assertions. You made the claim that IPENZ represented 10,000 engineers, who all had contrary viewpoints, to those expressed by John Scarry. This I very much doubt. This was a surprisingly naive comment from a minister, who was supposed to know what he was talking about. I thought, even the meanest intelligence, after a cursory study, would appreciate that IPENZ encompasses a wide variety of engineers, Chemical, Mechanical, Civil, which includes Roadage and Drainage, Bridge engineering, Hydraulic, Biotechnology, Electrical, and Soil Mechanics engineers, to name only some of the different disciplines, with comparatively few specialising in Structural Engineering design and construction………. This highlights the confusion that people have, in understanding an engineer's area of expertise. The term Chartered Engineer, unlike England, does not designate the area of ability, such as Chartered Structural Engineer, Chartered Civil Engineer, etc. which assures the public that these particular individuals are suitably qualified, in that particular discipline. Indeed, there are major penalties for any engineer masquera*ding *outside his/her discipline. You should bring this into line to prevent confusion, to not only yourself, but also the general public.*
……… ***IPENZ has a fiduciary duty, entrusted to it by the Parliament of New Zealand, to ensure the ability and performance of engineers****. This cannot be construed to mean passing the buck to extraneous committees or the DBH, becoming a mere sinecure, prolonging decision making and supporting inertia. IPENZ should be a leader, and speak with genuine authority. If they are unable or disinclined to do this, you, Mr. Williamson, should, immediately, act to remedy the situation.*

Clause 5.2 continued 'the Royal commission has become aware of instances in which engineers have undertaken and completed work in engineering specialties outside their area of expertise.'
This amounts to criminal fraud and where proven should be reported to the legal authorities for criminal redress.

In the section starting 'extent to which the candidate is able to do each of the following in his or her practice area is taken into account in assessing whether the overall standard is met.'
This is followed by eleven bullet points which are all important and glides on to the basic education of engineers.
Decisions should always be made on the basis of dialectic, which is the active investigation of the truth, by discussion and logical argument. Once this Socratic discipline is accepted, it is difficult to explain the a priori theories developed mainly by Professors Park and Paulay at Canterbury University, pertaining to the design of reinforced concrete multi-storied buildings, under an earthquake condition. These theories were adopted by the code writers and thus they became regarded as holy writ. These theories were based on experiments on single span frame assemblies, of a naked beam and columns, without any consideration of floors. I found this

approach to earthquake design completely nonsensical. It is neglected the major influence the floor would have on building frame. This approach represented an affront to all logic and a complete disavowal of reality. I had a flourishing practice, and pointed out to Prof. Park, that my clients were most unreasonable, as they demanded floors in their buildings. Research should always confront reality, and not ask the elephant to leave the room.

In September 1999, I wrote a paper entitled **Theory and Reality** for the SESOC Journal. This had the immediate effect, of incurring the wrath of academia, in the person of Prof. Paulay, as my views were opposed to the theories he was preaching and teaching, I found this quite understandable. Hell hath no fury like an academic scorned, especially when confronted with logic and facts, which challenge their cherished a priori theories. I will quote a few paragraphs from this paper:-
'Over 30 years, codes have changed with monotonous regularity, in both New Zealand and North America, in an endeavour to combat earthquake attack. Each new change is a tacit admission that previous codes have been found wanting, by actual earthquakes, and because of this, we can confidently expect many more codes to be written. In the early Seventies, there was a basic change in the philosophy of the SEAOC code. (The North American code- I was an Affiliate Member of SEAONC for many years) *The major change was not only the acceptance of reinforced concrete for high-rise construction, but also a change in philosophy from equivalency of energy absorption, to an adequacy criterion, as compared with structural steel. The code, at the same time, required all concrete space frames, to be designed as 'ductile moment resisting frames.' The introduction of the term of 'ductile,' and the code amendments, were obviously intended to preclude a brittle failure, at those points of high seismic moment end shear. Thus, from this point on, the major thrust, in subsequent reinforced concrete codes, was the adoption of this concept of 'ductile moment resisting frames.' It should be noted, that subsequent SEAOC codes, to their credit, have subsequently removed the term ductile, and re-placed it with 'special moment resisting frames.' (SMRF)*

As codes are incestuous, this concept has found its way from country to country, supported by the same inane test, of a 2-dimensional bent, being pushed back and forth, to presumably 'ape' the effect of an earthquake attack on a building, and then presented, in numerous papers, as the model for earthquake design. I have long maintained that this experiment has no relevance to the action of a building frame under seismic attack. In all my years as a consultant, nobody has ever asked me, to design a multi-story building, without floors. Surely it must be appreciated, that a three-dimensional building, complete with floors, is several light years removed from the concept beloved by numerous researchers, on both sides of the Pacific. The 'test' has been repeated 'ad nauseam' as if repetition somehow validated the illogical premises.

Even the words, 'ductile reinforced concrete' are an abuse of the English-language. To state the obvious, reinforcing steel is ductile, but concrete is not. The concrete will absorb load up to the limit of its tensile strength. This has enormous relevance, when considering the floor slab component of the T-beam. Only when the concrete cracks, can the rebar take any load. Obviously, when designing the beam / column joint, the beam is initially a T-beam. The flanges are a bay width, in the best case, with the numerous variances, due to possible saw cutting and/or precast floor slabs. It is also interesting to note that the 1997 SEAOC Bluebook, observes that the use of composite design of topping, together with the precast elements to act as a diaphragm, may very well be preferable to considering topping alone. Fortunately, I have always considered it thus, as topping lamination from well roughened precast elements is absurd. Much more research is required in this area, but tests without floors are futile. However, one thing we can be quite sure of, is that the floor will not neatly *crack, full depth, parallel to the beam resisting the lateral load, as in all the earthquake reviews, I have studied over many years, there is not one example of* such behaviour. *Perhaps the best example of cracked floors is in the now famous* ***Imperial County Services building (Oct '71, El Centro E/Q),*** *subsequently demolished, which augments the fact, that floors do not behave in the simplistic way described by our code.*

It is pertinent to point out, that in their excellent paper in the American Civil Engineer, (March '96) in an overview of Northridge, ***Chen and Yamaguchi****, discussed the effect that concrete slabs, over steel beams, had on substantially increasing beam strength and stiffness. Since designers had typically used only the bare frames, in their calculations, beams experienced larger forces than expected. They suggested that, because of the floor, the N.A. shifts up, increasing the stress on the bottom flange, and this may explain why damage to the bottom flange connections was so prevalent. Refer also to the Interim Guidelines Damage Classification, where this common* ***bottom flange damage*** *is noted. How much more positive are concrete floors, integral with concrete beams, in their effect on frame action? The tensile resistance of the floor will ensure that 'hinging' will always take place, at the soffit of the beam, adjacent to the column, not as observed in the standard bent test. Thus, even the basic theoretical lateral moment distribution model is incorrect.*
(Professors Chen and Yamaguchi were speaking from an overview, of the damage caused, by the Northridge earthquake, to over 200 steel framed buildings. No wonder, this confused the engineering fraternity, here in New Zealand, as they were asserting facts, which were contrary to theories taught in the hallowed portals of Canterbury.)
I continued,
The Pre-quake, the building sits at rest on its foundations, presumably, with all its floors intact, when a sinusoidal energy rocks its foundations, imparting vertical and horizontal accelerations to the building. The building response depends on whether it is a framed building, a shear wall building, or a dual system building. First, consider a framed building. The immediate reaction is elastic displacement, and the base shear must be a product of this displacement. The high stresses generated will cause cracking, and a longer period. The difference in the initial base shear and the theoretical base shear of the cracked structure reflects the energy dissipated by cracking. Further pulses cause larger displacements – larger periods - less theoretical base shear, until a building reaches the distressed state described in our code, with floors, beams, and columns, throughout all floors, massively cracked. The period of this severely damaged building is now taken to calculate the base shear. It is like getting on the tram, one section from the terminus. I have always regarded this concept as nonsense, and have never used it.
Please note; how the code arbitrarily reduced the Ig of both columns and beams, so as to obtain a ridiculously diminished T. Thus, it reduced design base shear to be ascribed to the building. With this concept, I believed that we had reached the nadir of stupidity, but I was wrong;- more was to follow.

After Richard Fenwick had the temerity to write a Guest Editorial to the SESOC Journal, I wrote, in the SESOC Journal, to both support him and to endeavour to wake engineers up to the real concerns that deserved their attention,
'I was impressed by the guest editorial, by ***Richard Fenwick*** *(Sept.2002) and the integrity he exhibited in stepping outside the academic cocoon, to examine the current rationale among engineering academics. I had patiently waited for some reaction from this article but the silence that greeted it was disconcerting, in view of the important matters that he raised. The current scenario he describes is one where practical design experience is disdained in favour of the writing of esoteric papers for export, based on a priori conjecture. Surely, if, as he*

describes, researchers are virginal as to practical site experience or even practical design experience, they live in a world of the abstraction, without the anchor of reality

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Mr Fenwick highlights two major areas of concern. First, he stated that recent research at Auckland university, show that the addition of the floor slab could double the maximum bending moments, sustained in the 'plastic hinge' zones in beams, and significantly increase the moment acting on columns. The only thing, I find amazing is that this is recent research. I have always included floor slabs in design, in assessing the strength of beams, relevant to columns. In March 1996, in an overview of the Northridge earthquake, ***Professors Chen and Yamaguchi****, discussed the dramatic effect that concrete slabs, over steel beams, had on beam strength and stiffness. They blamed this increased strength and stiffness, as the prime cause of the major damage, inflicted on over 200 steel framed buildings, in the L.A. area. They pointed out that, prior to Northridge, the stiffness of steel framed buildings, was assumed on the naked steel frame and the floors had been disregarded. Thus, a stiffer building attracted greater base shear. Significantly, they observed the major damage in the bottom flanges of the beams, adjacent to columns. This is a posteriori theory, based on evidence, which should be the only basis of research, academic or otherwise. Surely concrete buildings with floors monolithic with beams will behave even more effectively.*

Which brings us to Fenwick's major point. He reminds us that the 92 loading code, reduced the theoretical base shear, and thus reduced the seismic strength of multi-storey buildings. The theory is based on massively cracked frames, floors and columns, which will result in a large theoretical deflection and a greater T. ***(Ig for T & L beams, arbitrarily have their flanges reduced by 50%, then further reduced to 0.35 Ig & columns reduced by 0.6 Ig,)*** *Obviously, a smaller base shear will result in smaller seismic moments, less reinforcing, less strength, but large deflections, produced by the dramatic reduction of inertia of the beams and columns. This will produce a major P-delta problem, which will cause further moments and deflection, and then, this subsequent deflection will cause even more moments, etc. This is sometimes described as chasing one's tail. Fenwick's assertion that engineers ignore P-delta effects, in such a scenario, of artificially reduced shears, are alarming. To his credit, he previously covered this problem at some length, in the August/September 97 issue of the NZ Concrete Construction magazine.* (This is a relatively short article, entitled, **Significance of seismic induced P-Delta actions**, but it is of paramount significance, and should not be ignored)

He wonders where this theory came from. He is not alone. Northridge and Loma Prieta earthquakes, demonstrate that the vertical acceleration is a significant percentage relative to the horizontal acceleration. This fact, combined with the imponderable, of the energy dissipated by the rocking of the building on its foundations, militates effectively against certitude of a design method. In fairness, although I have seen what was described as the largest shaking table, in the world, in L.A., this could only impart horizontal shear. The fact must be faced, that we can never duplicate the sinusoidal energy, imparted by an earthquake to a building, in the university lab. and it is naïve to think otherwise. Rather, we should look to actual earthquakes for answers. Northridge was surely a wakeup call. ***Mark Fintel's*** *research, over many years, points to shear walls, as the optimum method of E/Q resistance Observations of earthquakes from Chile, (1960) through to Kobe, (1996) have encompassed a wide range of intensities, but throughout the whole gamut, shear wall structures had behaved very well. (Refer to Mark Fintel's brilliant paper,* ***Performance of Buildings with Shear Walls in the Last Thirty Years,*** *(in the PCI Journal, May 1995****,)*** *He pointed out that in all the earthquakes he has observed, starting with Skope (1963), right through until Armenia (1988), not a single concrete shear wall building had collapsed, while out of the hundreds of concrete* *structures that had collapsed, most suffered excessive inter-story distortions, which caused shear failures at columns. Even where framed structures did not collapse, large distortions, caused significant property losses. Why then, in our present code, do we base the building period on a massively damaged building?*

The ad hoc earthquake reconnaissance committee of SEAOC, after the Loma Prieta earthquake (1989), *concluded that for concrete buildings up to 10 stories the Blue Book equation for the fundamental period of buildings was quite accurate. (T= 0.073 h ¾)*
Consider the '89 Loma Prieta E/Q –7.1 on the Richter scale –
Maximum fault slip 7.5ft. (Horiz.Comp. 6.2ft. Vert. Comp. 4.3ft).
10-storey moment frame- T code formula= 1.11sec
(Est. T = 1sec from CSMIP Strong Motion Records)
It is pertinent to compare Northridge, 6.7 Richter and Kobe, 7.2 Richter.
The NZ Red Book *postulates a 10 storey building (ductile frame) with T> 2 seconds, (After badly miscalculating SEAOC formula as 1.63 seconds, - should read 1.08 seconds, which would be more relevant to the example, over.) and Ultimate Limit State cracking, based on the heavily cracked beams, floors and columns. Then for the same building, now considered as of 'low ductility' it is stated that cracking allowance is the same as the 'ductile' building! What nonsense is this?*

Briefly consider design drifts. NZ Code –maximum inter storey drift –0.015 for hn > 30m (0.015/µ=6 =0.0025)
SEAOC (*fundamental period > 0.7 sec)-drift shall not exceed 0.03/Rw - Or 0.004 storey height (Rw for Special Moment Resistant Frame = 12) 0.03/12 = 0.0025*
This is the storey drift limitation for the design lateral force. Then lateral defection must be checked for **Deformation Compatibility**. *The purpose of this is to check the lateral frame deflection with elements not considered part of the system for a major earthquake. Of particular concern were partitions, window frames and sufficient seismic gap to prevent pounding to adjoining buildings. This was judged to be 3(Rw/8) times elastic deflection, or 0.015h Thus, although there are marked similarities, their purpose differs. To summarize, for SMRF(Special Moment Resistant Frame) 3x12/8=4.5 x (0.0025)= 0.01125 and for* OMRF (*Ordinary Moment Resistant Frame)* 3x5/8 =1.875 x (0.0025)=0.0047
Clearly, the difference between the two codes is that one assesses the building period, from massively cracked beams, columns and floors, while the other uses a code formula, which has proved accurate up to 10 storeys. I have grave reservations about the NZ code approach

Back as September 99 in a paper I authored to the SESOC Journal, I tried to point out that the T values that engineers were using to ascribe the seismic shear to their buildings was dangerously an error. I referred to the original suggested value in older codes. *'No doubt, there are those who would consider T= 0.1 N as quaint. At the risk of introducing facts, consider the Ad Hoc Earthquake Reconnaissance Committee report, after the Loma Prieta earthquake.*
Date Oct 17th '89, -- measured 7.1 on the Richter Scale. -- Hypocenter 11.5 miles deep on the San Andreas Fault. -- Maximum east-west acceleration 0.44g
A summary of 13 buildings revealed that, for buildings less than 10 stories, the Bluebook equation 1-3, for fundamental period, was quite accurate, for structures other than shear wall buildings.
Equation 1-3, is a more sophisticated formula than T = 0.1 N, i.e.T = Ct (hn) ¾ (in feet)
Consider 10 floors at 12'-0" = 120 '-0" (36.4m)
Ct (RC frame building) = 0.03 therefore T= 0.03 (120) ¾ = 1.08 sec As we are not making watches, 0.1 N = 0.1 (10) = 1 sec would seem near enough

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Please note Red Book error *T= 0.11 (hn) ¾ (hn in metres)*

i.e. T= 0.11 (36.4) ¾ = 1.63 sec

Should read; T= 0.073 (36.4) ¾ = 1.08 sec.

Emboldened by the theories, in Aug. '98 a thick '**Red Book**' was presented to the engineering fraternity, by the Cement & Concrete Ass. Of NZ entitled, **'Examples of Concrete Structural Design to NZ Standard 3101.**' This was editored by Messrs Bull and Brunsdon, and was modestly presented as the design guide for framed and shear walled buildings. It was far from impressive. The framed building of 10 stories, studied in depth, was finally assessed to have a T (period) of 2 seconds. As I have previously pointed out in a paper to SESOC, the author in trying to interpret the SEAOC Code made a basic mistake, and the result he obtained was a T of 1.63 sec. when it should been 1.08 sec, which my old maths master would describe as a howler! This resulted in a decrease of shear to 70% of my result, but he went further, and finally arrived at a T of 2 sec. The Commission should read this, as Richard Fenwick will confirm what a massive difference this would make to determine the base seismic shear. It introduced to design engineers, to take to their bosoms and thereafter use it as a guide to all future endeavours. Forgive my sense of the ridiculous, but I rang the Concrete Association and asked for a refund, and explained the reasons why. They were not amused;-'the deep slumber of decided minds.' I am unaware of any correction ever being issued. At this juncture, I lost all credence.
At the risk of introducing facts, consider the SEAOC Report that stated after the San Andreas earthquake, (San Francisco,) where a summary of the 13 buildings revealed that for buildings less than 10 stories, the Bluebook equation 1-3, for fundamental period, was quite accurate, for structures other than shear wall buildings. Facts should override theory. This report was published in Oct. 17, 1989. The earthquake measured 7.1 on the Richter scale. Please note, the earlier NZ codes limited the maximum T, for any zone to 1.2 sec.

Clearly, Albert Einstein was correct, when he observed 'Unthinking respect for authority is the greatest enemy of truth.' Might I have the temerity to suggest, that the Commission is burdened with a major problem. It is reasonable to presume that a number of buildings have been designed and built to the code, with encouragement from the Red Book. I have outlined my concerns, over time, in a clear, logical manner. However, as a consulting engineer, my interests have always been designing buildings, rather than writing esoteric papers. I wrote simply to record the truth as I saw it, but I realised, that it was clearly impossible, to convince people whose minds are full of pre-indoctrinated beliefs, and it would be childish to believe that those in authority, and those engineers who had utilised these theories in designing their buildings, would docilely recant, as ego and reputation would cloud out reason. Codes should never be regarded as holy writ, as they change with monotonous regularity. **The blind adoption of codes does not absolve the engineer from employing logic, reason and common sense.** I must confess that throughout my career, after much research and reading, I have adopted a conservative approach to the allocation of seismic shear to my buildings. For this, I make no apology, as I developed sound ,efficient, economic, precast concepts, involving NMB's splice sleeves, (invented by Al Yee) which I brought into this country. Further, early in my private practice, I was responsible for both the architectural and engineering, as I realised the relevance that sound building concepts had to design. Indeed, unless a fairly symmetrical plan form and structural frame methodology is initially conceived, as is the basis of the NZ code, all else is fatuous nonsense.

Any concession to the irrational, introduces a stream of irrationality. Theories propounded by Canterbury, and adopted by our codes, concerning beam/column joints have been disproved by the research of **N.W. Hanson**, a prominent American researcher, by actual tests, over 40 years ago, and presented in a paper at the Lake Tahoe SEAOC convention in 1970. I have made this known to academia, but it is difficult to talk to closed minds. To compound this theory, it was further assumed, that there would be no reinforcing steel slippage through the joint, even at yield, and that all seismic joint shear must be taken by an impressive amount of stirrups and ties that, in many instances, precluded the adequate placement of concrete, which makes erudite discussions of bond through the joint, ludicrous. However, Mr. Fenwick, himself, has tested 500G Reidbar reinforcing, in T assemblies at Auckland University, to demonstrate bond failure, through the joint. *Quote; 'using the method suggested by Park (5) for assessing the available ductility from the accumulated ductility gives a value of 2, which is well short of the value of 6 that is required by the Concrete in Loadings Standards (1,6) for this type of construction*. This should be read in context with the whole report, but why has it taken all this time to debunk the Canterbury inspired code concept? Also, it is interesting to note, that Bob Park has pointed out, in his Sept. 2002 SESOC paper that in beam/column joints with code allowable db/hc, ***'some bond deterioration is inevitable and should be accepted**.'* <u>This is a very significant reversal to all previous dictates</u>. Hanson's tests pre-date this research by over 30 years. Mr. Park hypothesized that the reinforcing, at the remote face of the column would change in compression to tension. This is not Hanson's view, after testing both mild and high tensile steel. Thus, with the probability of bond failure within the beam/column joint, finally acknowledged, it follows that the shear steel in the joint cannot be developed. The diagonal concrete strut will resist the shear. Priority must then be given to confinement steel, top and bottom of columns, and carried through the beam. This will result in a dense well-compacted joint, which is of paramount importance. The code should be re-examined with a more sensible approach adopted**. So much for years of university 'research'.** I could not, on the basis of sound design, comply, with these requirements, so I developed a completely new methodology, (i.e. precast beam/column assembly in large T-units, poured into steel moulds, lying on their side, so that vibration was, at most, only through 400 mm.) These units were then erected using NMB splice sleeves. Thus, I could satisfy myself that we achieved strong compact 40 MPa concrete, despite the inclusion of code dictated shear steel. Thus, I must acknowledge the positive side, as Canterbury did me a back –handed favour, as I went on to use this method on a large number of significant buildings.

If those in supposed authority keep repeating the mantra that NZ codes lead the world in seismic design, they show an alarming capacity for self-delusion, having developed the habit of ignoring inconvenient reality. Smug attitudes do not encourage open minds. It is imperative to address the major aspect of site supervision of contracts. I wrote a guest editorial in the SESOC Journal (Vol. 14 No 1 April, 2001) to bring to notice the alarming lack of adequate working drawings, little or no detail and lack of site observation; surely an alarming situation, but the silence was deafening. Our code still refers to Ductile Reinforced Concrete. America has long since changed its designation to Special Moment Resistant Frames (SMRF), which is far more to the point. Concrete is not ductile but reinforcing is. There is merit in thinking clearly. IPENZ should set out a minimum of standards and an adequate minimum design fee structure, so that the standards may be achieved.

The Royal Commission notes that there have been very few cases of engineers being disciplined for the most severe of infringements-the performance of engineering services in a negligent or incompetent manner. This could be due to the inherent difficulty in demonstrating poor professional performance (e.g. the collapse of the building) the lack

of disciplinary action could reflect, will be a cause of, an apparent reluctance by engineers to publicly criticise each other.

I am a loss to understand from where you gleaned these impressions. Is it possible, that IPENZ are in complete denial**?** I have personally written, at length, to Presidents and boards of IPENZ, on three occasions, and it was a complete wasted exercise. Dr Charles Clifton, Mr Colin Nicholas, Mr John Scarry and I, met with Minister Williamson to report the performance of buildings which gave us great concern. His reaction was '*why were not some of these people in jail'* and *'he would get to the bottom of it!*'-We were all singularly impressed, but what is witnessed was**- Full of sound and fury signifying nothing**! He may have been placated by the CEO of IPENZ, as he gave intimation of in his TV program with John Scarry, but I would be very hesitant to believe this of the CEO. Further, there is John Scarry's Open Letter to the profession. If only a quarter of the cases that he referred to, were accepted and investigated, these would constitute a very alarming scenario for the engineering profession, and the public generally. I know it scared me, but he was largely ignored.

As I considered that these matters were too serious to be simply discarded, and reminding myself, that IPENZ has a fiduciary duty to the Parliament, to maintain the standards of engineering, I persisted with a letter to the Deputy Chief Executive Dr Nicki Crauford.
Q. You state that exists a disciplinary process,' ***and should examples of incompetence. negligence or unethical behaviour comes to light. an investigation will be undertaken****.'*
I accept that you believe this, most sincerely, and hopefully this situation will pertain on your watch. Unfortunately, it has not been my experience with IPENZ. I have written very comprehensive letters to previous presidents, namely Mr. Peter Jackson on 6 June, 2006, received his reply on 8 June, and I wrote again on 16 June 2006, and to Mr. Jeff Jones on 11 April 2008. Although I had been prompted to write by engineers and members of the building industry, I can only describe my endeavours, to communicate my concerns to IPENZ, about the abysmal state of the building industry, as a complete waste of time. Mr. Jackson at least had the courtesy to write to me and explained to me that he was a ***'non –structural****' person. Mr. Jones did not even provide me the courtesy of a reply. I admit to being most annoyed at the time, as the matters raised were of singular importance for those engineers, who are engaged in the structural design of buildings. I will quote the first paragraph of my communication, so that you may be the judge.*

'In the remote chance it has no yet been bought to your attention, I write to inform you that in ***the NZ Herald of Saturday the 17th November, 2007****, there was a major article, comprising four pages, by a* ***Mr Simon Collins****, concerning the inadequate structural design of a number of major buildings. The article begins, '***Design faults brought Mall safety changes – Poor engineering work risks producing unsafe <u>public buildings</u>.'** *The relevance and accuracy of his report was assured by the measured, professional comment by engineers,* ***Messrs Davidson, Tyndall, Jacobs, Nicholas, Fenwick and Scarry,*** *who confirmed the various facets of this alarming scenario. By speaking out, they installed some modicum of integrity to the structural engineering profession.'*

It should be noted that, ***Mr. Simon Collins*** *is an award winning journalist, and was awarded the prize for Excellence in Engineering Journalism, and also the Qantas Media Award, so he is no hack .What did I do wrong? Wasn't it plain enough? I would have thought, considering the content of the whole letter that it would raise serious concerns to even a person whose experience may have been other than structural engineering, or could it simply be a case where reality is not permitted to intrude into a closed mind.*

Quite naively, as I now recognise, I wrote to the Minister Williamson in an endeavour to set out points that he should raise with IPENZ, in the simplest form I could construe, which took the form of my questions to Dr Nicki Crauford and her answers.
Q. What does having confidence in the DBH' have when they employ outside engineering consultants to do reports – even the recent important report on the Christchurch CTV building, -- which suggests a mere sinecure. Refer also to the Stadium Southland, yet to be published report. I have since been informed by her, that, *'A report on the Stadium Southland collapse has unfortunately been delay(ed) as a result of the consulting engineers being diverted onto Canterbury earthquake matters.*
It would be an interesting exercise, for you, Mr. Williamson, to determine, in detail, what the DBH staff, of several hundred, actually do. What enables IPENZ to recommend extraneous structural engineering experts, when they have only limited acumen, in this particular area? Is it true, Mr. Williamson, that the Government restricted the investigation, to the narrow confines of these 4 CBD buildings; - surely not! On their record, I do not expect IPENZ to discipline any of the design firms, responsible for those projects, we brought to your attention in December, 2008. Remember, the ones which got you so riled! Such as the Vector Arena which was, during construction, in such a parlous state, that everyone was evacuated from the site, the rail-train stopped, while urgent remedial work was carried out by a few brave souls to fix the major problem, and prevented a major foul-up. What changed your mind? Pressure from above? The facts about the Arena are readily available. In fact, one of the group, gave you an eyewitness account. But hark to the reason that no blame was attributed to either the engineers who designed the building or the firm of engineers, who checked it. From Dr. Crauford, no less, in a reply to me;-

In the case of the Vector Arena, which you mentioned in your letter, the Building Consent Authority, commissioned a report which concluded there were <u>systemic</u> issues involving a significant number of engineers, and that none should be personally investigated. Hence no individuals were identified to us.

Mr. Williamson, although this fiasco did not occur on your watch, you promised us, to do something about it. How I hate this term '***systemic,***' I am aware that the term 'systemic issues' has come to be used as a catch-all myopic excuse, when the Oxford dictionary definition is 'of a bodily system as a whole, not confined to a particular part.' As you are, no doubt, aware, Socrates observed that the beginning of knowledge is the definition of terms. Note that the report was commissioned by the Building Consent Authority, not IPENZ, whose fiduciary duty it is to examine such problems. The BCA has its own barrow to push. So, are we to accept that that 'systemic issues' absolve 'a significant number of engineers' both from the design engineering firm and the firm engaged in the checking the design, should not be' personally investigated.' Do you support this nonsense?

My question to Dr Crauford;-
In an endeavour to get to the truth of one of the major causes of the catastrophe that has occurred in Christchurch, I think it would be rewarding for both IPENZ and the Commission if they would investigate a 120 page (1991) report, which was EQC funded. The lead author was Don Elder, who in the report, basically concluded that Christchurch was at very significant earthquake- shaking risk. There is an excellent article in the September (3-9) Listener, which summarises these conclusions. These give the lie to the assertion that no one was aware of the inherent dangers that existed within the Canterbury Plains. It stated that the strata underneath the city will amplify the earthquake shaking and concluded that the greatest concern was liquefaction. Please take the time to read the full article, as it gives a commendable overview of what has happened in Christchurch. Elder could not understand

why conclusions of the lengthy report were not included in the **1992. Loadings Code**. Knowledge and acceptance of the major problems revealed in this report, **which was produced 20 years ago**, could have prevented at least some of the huge problems that confront this beautiful city, and could have even saved lives.
Her reply;- *We are familiar with the report, primarily authored by Don Elder, to which you refer The report was considered by the 1991 Loadings Standards Committee.* ***CNS Science*** *offered an alternative view which was subsequently accepted by the Standards Committee.* This conclusion was most unfortunate for the City of Christchurch. **Citizens were misinformed, and the resultant Code, deluded the building designers. As there was not even a suggestion of regret, and since I believe this whole question is so salient to the future of Christchurch, I tried again;-**

Q. The fact that the nature of the strata underlying Christchurch and the prediction of liquefaction was very competently set out in 1991, by a 120 page report, commissioned by the EQC, and was ignored, and not included in the 1992 Loadings Code, is simply unbelievable. That this report was blithely disregarded by the code writers, as you have informed me, in preference to the **GNS Science** views, was most unfortunate. This was further augmented by a 1996 documentary **'*Earthquake*'** and was also subsequently shown on You-Tube, with some very damning observations. Refer, also, to the comments made by **Sir Kerry Burke**, in the Christchurch newspaper **'Star'** on the 11 March.' His remarks have singular impact, as he was the **Environment Canterbury Chairman**, and he stated that liquefaction occurred pretty much as the **ECan report** had predicted. Collectively, one would have thought, these reports were difficult to ignore. Decisions such as these have consequences, and accrue responsibility.
No answer was the stern reply! Apparently no one was to blame, so nobody had to accept any consequences. What childish arrogance! A profession implies real parameters and professionals are those who bear responsibility for their advice.

As I have previously pointed out to the Commissioners in my previous submission, my concerns about the design of Shell beams. Quote*;- I would be remiss, not to point out that I was and continue to be surprised, at the ignorance revealed in emulating some of my methods particularly Shell Beam design, which in my opinion, as the co-author of the original idea, believe are being widely used improperly, and sometimes dangerously, by those who seem to have little understanding about composite action, vertical and longitudinal shear in the treatment of the positive and negative moments in their design. Some competent authority should immediately redress this situation*.
Briefly, the Shell Beam concept was conceived by Mr Esli Forrest and I, in 1958/59. I have used them successfully for over 35 years, in many buildings, including multi-storied buildings valued at millions of dollars. It was with amazement that, after using them successfully over a long period of time, I learned the system had been referred to Canterbury for testing their ability under earthquake conditions. The usual experiments were done on naked frames, the system OKed, and then straight into publication mode. The paper appeared in the ***PCI Journal July/August 1986 edition authored by Park and Bull.*** When I read the paper I was appalled.-I would not dream of using Shell Beams as was set out in this paper. Without prior reference to either Mr Forrest or myself, they committed themselves to print;- -the arrogance of ignorance! **Alfred Yee**, a world class American consulting engineer and a renowned expert in precast concrete, told me he had discussed the matter with his great friend **T.Y. Lin**, a world class authority in prestressed concrete, and they both wanted to know my approach to the design of the mid-span moment. I discussed the matter with him, and he said he was impressed, with the method which I always used. They had both had been concerned that somebody would be foolish enough, to simply take the Ultimate Moment of Resistance of the strands. I was gratified by their interest and their endorsement of the concept. Sadly, I have

observed *Shell* beams, severely distressed, in a public car-park building, (which I reported to both IPENZ and SESOC) and on numerous occasions, continuously propped, during construction, along their length and down, through three, and even four floors, to ground, with the concomitant costs, in both time and money. Obviously, a complete negation of the concept that Esli Forrest and I, developed so many years ago. Why use them as expensive boxing, with a plethora of propping and waste time, money and speed of construction? Ignorance may be bliss, but clients should not have to pay for it.

The Commission asks for comments on the efficacy and efficiency of the conduct of the engineering, in respect to be in interactions between structural engineers and geotechnical engineers.
The retention of a top geotechnical engineer is of paramount importance in the design of all multi-storied buildings, and indeed, even single-storey buildings to be founded on any questionable site. I have been most fortunate in this regard, as early in my career I met with Mr Ralph Tonkin. I was working at the time as a design engineer for a firm of engineers on the ANZ bank building in Queen Street, Auckland, at that time the highest building in Auckland. I was tasked to redesign the foundations. Unfortunately we were faced with basalt rock, at the Queen Street frontage and then into Waitemata series at the rear. But for Ralph, we could have another leaning Tower of Pisa. I developed and retained great respect for him, as I considered him brilliant. Thereafter, in private practice, I invariably used the firm of Tonkin and Taylor. Throughout my long career, and my long association with them, I developed confidence in them, and though dealing with varying personnel, over the years, I have never been disappointed in them. If you will forgive the pun, the firm had good foundations.

Finally, I wish to reiterate, that which I pointed out in my first submission to you, and that is since our code was initially developed from the SEAOC Code pertaining to Zones 3 & 4 in California, their requirements for site supervision should also be adopted. Quote ***'a specially qualified inspector under the supervision of the person responsible for the structural design, shall provide continuous inspection of the placement of reinforcement and concrete and shall submit a certificate indicating compliance with the plans and specifications'*** If this had been previously adopted, some of the uncertainty surrounding the CTV building would have been dispelled.

May I thank you for your patience in reading this submission.

Yours sincerely,

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