Gentleman,

I am prompted to proffer the following points for discussion, regarding the effect that the Christchurch earthquake has had on the four buildings you have been assigned to examine. My motivation to do this is because of the appalling death toll, with particular reference to my neighbour's two close relatives in the CTV building and mindful of the onerous task that has been entrusted to you. While I know and respect Mr. Richard Fenwick, I have not met Sir Ron Carter for some time and have no knowledge of Justice Cooper, thus it is appropriate to present a brief overview of my previous experience, in structural design, so that you may judge whether my comments are worthy of your consideration.

My name is Carl Robert O'Grady. I am a retired Consulting Structural Engineer, a Fellow of the Institution of Structural Engineers (London) a Chartered Engineer (London), MIPENZ (Retired), and for many years an Affiliate Member of the Structural Engineers Association of Northern California. (SEAONC)

I conducted a private practice, under my own name in Auckland, for the greater part of over 42 years. I retained a junior partner, from mid 1984 to mid1993. After recovering from two major operations, I returned to private practice, and was again engaged in the design of major projects, including a 16 story apartment building (which broke new ground as it was designed in high-strength precast concrete to comply with the 'ductile' provisions of the code.) I have designed an impressive number of buildings, including many multi-storied buildings both in Structural Steel and Reinforced Concrete. Frankly, I did not fully appreciate the extent of the work, until Mr. Al Yee of Alfred S, Yee & Associates, a world famous engineering practice, asked me to prepare a Career CV, with a view to a possible joint design venture in China. It would be fair to say, that I have earned a considerable reputation in the field of precast concrete, both here and overseas. This stems from the early days in the office of A.S. Miller, when together with Mr. Esli Forrest; we developed Shell Beams, of varying depths, to be used with simple channel slabs. Over the years, I have refined my approach, using various concrete floor slabs, including an untopped floor slab methodology. (which I patented) As I realised, the future in precast concrete was to devise some reliable method to join walls, columns and large Tee units together, I had the great good fortune to meet Mr. Al Yee, on an overseas trip, and I learnt of his NMB splice sleeve, for various sizes of reinforcing steel, used in conjunction with high strength grout. After seeing the results of the testing that had been carried out, and their inclusion in major projects, I immediately appreciated that this was exactly what I was looking for as it was, in my opinion, the result of a genius at work. I am in his debt. He had sold the patent to Japan, so I included them into my designs, and arranged to import them and introduced the methodology to N.Z.

As we have been informed that the recommendations from the Commission include 'any measures to prevent or minimise the failure of buildings due to earthquakes,' 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 Beams design, which in my opinion, as the co-author of the original idea, believe are being widely used inappropriately, and sometimes dangerously, by those who seem to have little understanding about composite action, vertical and longitudinal shear and treatment of the positive and negative moments in their design. Some competent authority should immediately redress this situation.

As I spent 3 1/2 years at school in Christchurch, I have always entertained fond memories of the city and am enormously saddened by the devastation of buildings by earthquakes in recent months. I offer the following thoughts in the hope they may have some relevance in the reinstatement of this beautiful city.

Initially, I had reservations about the composition of this Royal Commission, but after reflection, I can see the merit in the mix of differing expertise:-Mr. Fenwick for his theoretic scholarship, and also the acumen and courage he has shown over the years, Sir Ron Carter, for his overview of large firm politics and the actual administration and execution of contracts, and Justice Cooper to provide balance and an unlaundered judgement on the issues, where the only criteria are design methodology and contract execution, divorced from overt allocation of blame. The prime goal then becomes a search for truth, which I totally endorse.

Firstly, and most relevantly, the Commission should urgently address the fact that the nature of the strata underlying Christchurch and the prediction of liquefaction was very competently predicted in 1991, by a 120 page report, commissioned by the EQC, and was ignored. The lead author, Mr. Don Elder, then with the firm of Royds Garden, expressed his surprise that this report was not included in the 1992 Loadings Code. On balance, it simply unbelievable, that this report was blithely disregarded by the code writers. I have been informed by Dr.Nikki Crauford, Deputy Chief Executive of IPENZ, that, the 1991 Loadings Standards Committee preferred the GNS Science views rather than the Royds Garden report. This was most unfortunate. This was further augmented by a 1996 documentary ‘Earthquake’ and was 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’ on11 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. It is alarming to read the article (www starcanterbury.co.nz) Also note that the report was based on the result of 600 drill holes, thus was quite comprehensive. Collectively, one would have thought, these reports were difficult to ignore. Decisions such as these have consequences, and accrue responsibility. One cannot overemphasise the importance of retaining highly competent foundation engineers, in the design of major buildings, or for that matter, any building, where the ground is suspect. It would be relevant for somebody in authority, to rigorous question of Council as to why housing subdivisions were allowed on rumoured questionable land. (Refer Sir Kerry Burke’s remarks.) This was of salient importance, not only to determine foundation pressures, but also to determine how the seismic sinusoidal pressure wave enters the building, be it the piles, pads or foundation beams. If piles go through cohesionless soils going to bedrock, where is the point of entry and what moment must be sustained by the piles and the ground beams? This always had enormous relevance to the conceptual design of the building. Further, it should come as no surprise that the vertical component of the E/Q energy wave is, of the order of 70% of the horizontal shear as this is experienced and both San Francisco and L.A earthquakes.

Secondly, from information received from SEAONC (Structural Engineers of Northern California) over the years, pertaining to various earthquakes throughout the world, I have gleaned an extensive overview of structures under seismic attack, but I find it difficult to explain the Christchurch earthquake period, of both reinforced concrete and steel buildings, and the after shocks over this long period. The longevity and close proximity of what are constantly referred to as after shocks are beyond my own considerable study of earthquakes. However, I think it would be rewarding to consider the overview of the Loma Prieta earthquake (San Francisco) in 1989, published by the Ad

Hoc Earthquake Reconnaissance Committee of the Structural Engineers Association of California. I believe it has relevance to the Christchurch experience. In the Marina area of San Francisco, sand boils which are evidence of the liquefaction phenomena, were much in evidence and liquefaction did occur, resulting in the loss of foundation support in numerous structures. The report states that liquefaction can occur when saturated fine grained unconsolidated cohesionless soils experience cyclic shear stress. They also state that soft soils significantly amplified base rock motion and that there is a definite correlation between damage patterns and soil types. This, I contend, has immediate relevance to Christchurch, as I understand the underlying strata consist of shingle silt and sand, with a water table not far from the surface. There is also evidence of liquefaction in Kobe, Japan, and even more importantly, Mexico City. Mexico City, as you are no doubt aware, was built on ancient lake bed. Reports and opinions about the increase in amplification vary from a margin of 4 to an even larger number. The damage was massive. For this reason, the rush by SESOC to contemplate a change in the requirement for code base shear in Christchurch area is premature. The immediate consideration is what remedial action is to be taken with the underlying strata, and no buildings should even be considered without extensive vibro- compaction.

Thirdly, another stern warning from San Francisco is the major damage caused by buildings pounding against each other. I have been informed that the Hotel Grand Chancellor, subject to demolition, has experienced major damage, to a column, where it has pounded against the adjacent car-park building. This could also be a cause of damage to some of the medium rise buildings, currently being demolished, as there was in San Francisco. This lesson should be foremost in the minds of those who contemplate the rebuilding of the city.

May I preface my comments about specific buildings, with the overriding consideration that where liquefaction is present, the amplification of the sinusoidal energy wave imparted to the building can be in excess of four times of what would normally be expected? While I do not have knowledge of the strata at specific sites, which would vary, it would be most unfair to judge any engineers involved in a particular design, as the nature of the strata could increase the ground shear conditions, imparted to the building by a significant amount. I wish to comment only on design principles.

I am astounded by the extent of the damage to the CTV building, which I seen on fairly extensive television coverage. Due to the aftermath of the earthquake, the 5 floor plates have collapsed into rubble, while the Lift/Stair tower remain erect, with, what appears to be, a combined shear tension failure, in the slab adjacent to beam in front of the Lift/ Stair. (See T.V. coverage) It is reasonable to conclude that not much seismic shear was distributed to the very stiff 'C' wall at the rear of the Lift and Stair. This is not surprising as the majority of the wall is divorced from the floor slab. (Refer the dimensioned plan of this area.) The length of the wall from the inside faces of the return walls is 11.500. However, the openings for the Lifts and Stairwell preclude any shear distribution, to the wall. To further compound the problem, there is another void, for services, in the adjacent room, leaving only 2.500 of floor remaining to transfer the shear. SESOC has remarked in its preliminary report to closely examine the stability of the return Flange walls, but neglect the obvious, where one Flange wall is without restraint because of the void created by the Lift Shaft.

The seismic shear in the other direction is presumably taken by the Cross Shear Walls, hopefully in concert with the peripheral framing. Because I have been influenced by the SEAOC Bluebook, the necessity for drag bars in a collector beam or poured in situ floor slab is readily apparent to me, to transfer the diaphragm shear into the wall, particularly where the walls have no restraint from floor slabs. (Refer SEAOC Bluebook -C407- Page 226 of the Oct. 1966 edition) Also note that the reduction factor for chords and collectors should be taken as only 0.60. I saw no evidence of drag bars, in the TV overview, but hopefully they where supplied and lapped well into the floor and simply yielded and broke off.

If one surmises that the basis of the design is that the seismic shear is taken by shear walls in the cross direction, the enormous difference in stiffness of the 'C' wall, at the back of the Lifts and Stairs, and the 5 m wall at the other end of the floor plate, (which collapsed) would create a large eccentricity, as the centre of mass and the centre of stiffness would be large – (together with the additional Code required +0.1 B.) This would throw undesirable torsion into the diaphragm, walls and frames. Where a Dual System is used to resist seismic shear in Zones 3 &4 in California, the walls must be designed to take 100% of the shear and it is mandatory that the frames are designed to take 25% of the total shear. While this may be construed as a belt and braces approach there are good reasons for this precaution. This is not a requirement in the New Zealand codes, but not to prescribe any lateral shear into the frames would be naïve, in my view. This must not be construed as a criticism of the engineers involved, as the Code did not and does not, adequately cover this aspect of design. Nonetheless, I consider this approach, to be salient to sound design. This is a very important aspect to both conceptual and actual design. As an overview, I wonder when it will be adequately realised that the architectural design dominates the structural viability. If this is not appreciated by both architect and engineer, and a sound symmetrical plan form is not devised, all else is fatuous nonsense.

From the news reports, there will be recommendations from the Commission on:

- Any measures to prevent or minimise the failure of buildings due to earthquakes;
- The cost of any measures; and the adequacy of legal and best practice requirements for building design, construction and maintenance, in relation to earthquakes.

I believe a change in current design attitudes and a readdressing of some major current NZ code directives, would provide a marked improvement, and would assist young engineers who have been taught in university, what to think, not how to think. There follows my suggestions:-

The codes should not be regarded as holy writ. They should not be devised as cook book codes to protect the incompetent. I regard codes as crutches and have always adopted the advice of T.Y. Lin, who I greatly admire, to 'rather than blindly follow the codes of practice seek to apply the laws of nature.' The blind adoption of codes does not absolve the engineer from employing logic, reason and common sense. Richard Fenwick is aware of my adverse opinion of the dominant Canterbury University approach to structural design, which has found its way into current codes. I have made my views public in the number of papers that I have written to the SESOC journal. My views have encountered the wrath of some academics. Hell hath no fury like an academic scorned, especially when confronted with unassailable facts to negate their a priori theories. These theories 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. NZ codes did not recognise bond slippage at high stresses and the actual mechanism that develops through the knee joint. The antithesis of Hanson's test results gave rise to badly designed joints, with such steel congestion that it became a near impossibility to place concrete around the reinforcing. I could not, on the basis of sound design, comply, 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. It is pertinent to note that the frames utilised by universities to propound these theories were without considering the floor slabs. I had the audacity to point out that all my clients demanded floors in their buildings. It should always be realised, in university research, that it is a sound idea to confront reality

Emboldened by the theories, in Aug. '98 a thick 'Red Book' was presented to the engineering fraternity, entitled, ' Examples of Concrete Structural Design to NZ Standard 3101,' This was modestly presented as the design guide for framed and shear walled buildings. It was far from impressive. The framed building of 10 stories 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 fundamental mistake, and the result he obtained was a T of 1.63 sec. when it should have been 1.08 sec. 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. 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.

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.

A final important aspect to note, is that in Zones 3&4 in California,

'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.'

This is a far cry from what pertains on our building sites currently. The whole question is of paramount importance, and the Commission could achieve a major service to future building safety, if it can change the fee structure. Engineers may then provide adequate design, production of decent working drawings and site observation. (At least.) This will require Government action but you are tasked to produce meaningful reforms. Please refer to Veralum (Structural Engineer August 2011.) where an excellent overview of Status, Quality, and Fees, by Richard E Harris appears. He has made comparison of results through different countries. You can achieve what England has not, as you have the status and clout to influence Government.

May, I thank you for your patience in reading this submission, and offer you my sincere wish for success in your endeavours, and the courage to confront your arduous task.

Yours sincerely,

Carl R. O'Grady

C.Eng. (U.K.), Fellow I. Struct. E. (London)

M.I.P.E.N.Z. (Retired)

Chartered Structural Engineer (U.K.)

Phone/Fax (64-9) 5768756.

E-mail - carlogrady@vodafone.co.nz