Under	**THE COMMISSIONS OF INQUIRY ACT 1908**
In the matter of the	**CANTERBURY EARTHQUAKES ROYAL COMMISSION OF INQUIRY INTO THE COLLAPSE OF THE CTV BUILDING**

SECOND STATEMENT OF EVIDENCE OF JOHN BARRIE MANDER

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SECOND STATEMENT OF EVIDENCE OF JOHN BARRIE MANDER

1. My full name is John Barrie Mander. I reside in College Station, Texas. I hold the position of the Zachry Professor of Design and Construction Integration 1, within the Zachry Department of Civil Engineering at Texas A&M University.

2. I refer to my first statement of evidence dated 10 June 2012 for details of my qualifications and experience. I again confirm that I have read the Code of Conduct for expert witnesses and that my evidence complies with the Code's requirements.

3. I am providing this second statement of evidence to introduce the results of further investigation and analyses I have been completing. As far as my other commitments have allowed, I have followed the first two weeks of the Royal Commission hearing via the online live stream and archive. These further investigations and analyses have been prompted primarily as a result of evidence I have seen presented by other witnesses, in particular Dr Heywood and Dr Kehoe.

4. In carrying out my further analyses, I have worked closely with my former mechanical engineering PhD student, Dr Geoffrey Rodgers, currently a Post Doctoral Research Fellow at the University of Otago Medical School and an adjunct to the Department of Mechanical Engineering at the University of Canterbury. Dr Rodgers has primarily assisted with retrieving and collating the records from the four recording devices around the CBD that are referenced in the my report and running the computational model to generate the fatigue spectra.

5. The new evidence presented in my report helps supplement evidence presented in my first statement regarding side-sway as to why structures fail. The methodology and reasoning I present are not well known or widely understood. Much more work could be completed to further advance the analyses but time has not permitted this. The project should, however, be considered a work in progress.

6. My report is **annexed**.

7. Supplementing the information in my annexed report, further details of the timing, peak accelerations and locations of the significant earthquake events from 4 September 2010 up to and including 22 February 2011 are at **[BUI.MAD249.0502]**.

Dated this 9th day of July 2012

JB Mander

J. B. Mander

Progressive Damage Accumulation in Earthquakes within the Context of the CTV Building Collapse.

By John B Mander PhD, FIPENZ

Contextual Background

There are two key factors that lead to the overall damage effects on structures in earthquakes. The first is the *maximum response* displacement or drift that arrives from side-sway effects during shaking. The second, and often neglected effect, is the *duration* of the earthquake and the cumulative damage effect caused by the repeated *cyclic loading*.

Cyclic loading demands and their effects can lead to fracture or failure of key structural elements, and thus act as a trigger that will either lead to a lack of serviceability (such as, for example, excessively high floor vibrations) or a general collapse condition. Such phenomena come under the general category of fatigue loading. Fatigue can be considered in a disciplined way by separating the phenomena into fatigue capacity versus fatigue demand. For a safe operational condition, engineers need to check that the fatigue capacity exceeds the fatigue demand. This report focuses on the latter aspect in an earthquake engineering context. There are two types of fatigue demands that plague structures, high-cycle fatigue and low cycle fatigue.

High-cycle fatigue is the most well-known class of fatigue. It occurs under normal day-to-day operational conditions and is a common problem in aircraft and other mechanical structures that are prone to vibration effects. The number of cycles to failure for this class of mechanically engineered system is generally in excess of one million cycles.

Civil structures, such as steel bridges, can also suffer from high cycle fatigue. To provide fatigue resistance, civil engineers strive to keep the double amplitude stress reversals below a so-called *fatigue-limit* threshold. For steel, this can be in the order of 150 MPa. High cycle fatigue generally occurs where the stresses remain in the elastic condition.

Low cycle fatigue can also plague structures such as buildings and bridges under extreme loading cases such as in earthquakes. The number of cycles to failure is referred to as "low-cycle" because the material is commonly expected to be taken well beyond the yield stress or strain limits into the inelastic range of behaviour. Much work has been done in this area by the author and others. For example, see Mander et al. (1994), Dutta et al. (1999), and Dutta and Mander (2001).

Although structural engineers are not explicitly required by code to design for low cycle fatigue effects, designers are required to be aware of the ramifications of low cycle fatigue. For example, Clause C3.2 of the commentary of the loadings code NZS4203 states that the structure should be capable of sustaining four fully reversed cycles of loading without losing more than 20% of its (strength) capacity. This criteria, although not normative and only informative, has historically been understood in the New Zealand context as the expected cyclic loading demand one should consider in structural design.

The reason that this implied fatigue demand in clause C3.2 is informative and not normative is because the associated material codes such as NZS3101, provide prescriptive seismic detailing requirements that have been checked out by laboratory tests to ensure that the cyclic loading capacity of ductile elements can sustain more than the four complete fully reversed cycles of loading implied by the loading codes.

The problem arises when multiple earthquakes occur because engineers are often required to provide owners/insurers/regulators information of the remaining fatigue resistance. Such remaining life is often posed as a question to an engineer evaluating a constructed facility: "Can this structure survive another design-level earthquake?"

Fatigue Demand Analysis of the Canterbury Earthquake Sequence

This report seeks to quantify the degree of cyclic loading damage imposed on structures in the Canterbury earthquake sequence and to draw conclusions as to how the cumulative effects compare with the cyclic loading demands implied by the code NZS 4203.

There were 15 earthquakes greater than or equal to M5.0from 4 September 2010 to 22 February 2011. A summary of these records, in terms of their location and their peak ground acceleration (PGA) shaking intensity are given in Table 1. The Boxing Day 2010 earthquake, although less than M5.0 (at M4.9) is also included because of its proximity and effect.

Response spectra for the five earthquake events with the highest recorded PGAs between 4/9/2010 and 22/2/2011 are shown in Figure 1. Figure 1 shows that while all of these five major events have notably high PGAs (the spectral response at T = 0s), the spectral response near a design period of T = 1s is more varied. The 4 September 2010 and 22 February 2011 events have notable long-period content, whereas the other events have less content in this range. However, all of these five events have notable spectral response in the T = 1s period range which will produce ongoing cumulative demand on a structure with periods in the range of 1 to 2 seconds.

Figure 2 presents the spectra of the effective number of cycles for three different fatigue exponents. These three fatigue exponents provide different weighting on different displacement response amplitude cycles. These fatigue exponents correspond to different material classifications.

The cycle equivalence calculations are based upon inferring the peak displacement points throughout a time-history analysis at each change in direction of the structure and calculating an effective number of cycles for a given reference amplitude. This analysis approach is analogous to the root-mean square signal processing method, which relates overall variable signal amplitude to an average value.

Specifically, the cycle-counting approach is given by:

The effective amplitude, ε_i, at every displacement point can be calculated relative to a given reference amplitude, A_{ref}, where:

$$\varepsilon_i = \left(\frac{|x_i|}{A_{ref}}\right)^C \tag{1}$$

in which x_i = the i^{th} displacement point and C is the fatigue exponent.

The value of C is taken to be = 1 for concrete-critical fatigue, = 2 for reinforcing-steel critical fatigue (Mander et al. 1994 and Dutta and Mander 2001), and = 3 for structural steel critical fatigue.

The mean, m, of all displacement point can be determined from:

$$m = \frac{\sum_{i=1}^{N}\left(\frac{|x_i|}{A_{ref}}\right)^C}{n_{po\,int\,s}} \tag{2}$$

where n_{points} = the total number of points for that record. That is, $n_{points} = t_f/\Delta t$, where t_f = the final time for the record (the record length), and Δt = the time-step for that record. This mean-value can be transformed into an effective amplitude, based upon the integration of fully reversed sine-wave cycle. For C = 2, this analysis is the same as a root-mean-squared approach whereby the effective amplitude can be determined by multiplying the mean value by a multiplier B = $\sqrt{2}$. For C = 1, B = $\pi/2$ = 1.57 and for C = 3, B = 1.33. Therefore the effective amplitude becomes:

$$A_{eff} = Bm^{1/C} = B\left[\left(\frac{\sum_{i=1}^{N}\left(\frac{|x_i|}{A_{ref}}\right)^{C}}{n_{po\ int\ s}}\right)^{1/C}\right] \tag{3}$$

The effective number of fully reversed cycles at the current design period of interest can be determined from:

$$N_{cycles} = \frac{n_{po\ int\ s}\,\Delta t}{T} = \frac{t_f}{T} \tag{4}$$

where T = the natural period of the structure of interest.

Finally, the number of effective cycles at the reference amplitude can be determined from:

$$N_{eff} = N_{cycles} A_{ref}^{\ C} \tag{5}$$

This final result, N_{eff}, presents the equivalent number of fully reversed response cycles at the reference amplitude A_{ref}.

For the results of Figure 2, each orthogonal ground motion record for every available recording station (from CBGS, CCCC, CHHC and REHS) were simulated and normalised to a reference amplitude, A_{ref}, equal to the *spectral displacement for that specific record*. Therefore, the results of Figure 2, present the record-to-record variability in terms of record duration and distribution of response cycle magnitude. The results of Figure 2 are plotted on a log-log scale. The log-normal mean and ± one log-normal standard deviation (16th and 84th percentiles) are presented on the graph as red lines. The black lines represent the fitted trendlines to these lines. Therefore, these results give an indication of the record-to-record variability, indicating the range of equivalent fatigue cycles, relative to the peak response for that record.

Significance in the Context of NZS4203

It is considered to be of particular importance to compare the cumulative fatigue demand to the design code requirements, as in NZS4203. Therefore, in the subsequent analysis, the reference amplitude used for the cycle counting is the spectral displacement amplitude from NZS4203. The spectral displacement from the code, based on Zone B – Soft soil for Christchurch, with ductility, μ = 4, from Figure 3 of NZS3101 and uniform force reduction factor, $R = \mu = 4$ as per Newmark's well-known equal displacement factor. Taking peak maxima only and simplifying yields:

For short-period structures, $T \le 0.7$ s, $S_d = 0.12425\ T^2$

For long-period structures, $T \ge 1.2$ s, $S_d = 0.07455\ T^2$

For medium-period structures, $0.7 < T < 1.2$ s, $S_d = 0.1938\ T^2 - 0.0994\ T^3$

The results of this analysis for the three material classifications with fatigue exponents, C = 1, 2, and 3 are presented in Figure 3.

Figure 3 represents the equivalent number of design amplitude cycles for the Canterbury earthquakes. Three curves are plotted on each graph. The lower (blue) curve represents the number of NZS4203 design demand cycles experienced as a result of the Darfield earthquake on 4 September, 2010. The mid (red) curve represents the total damage done by the Darfield earthquakes plus the aftershocks prior to 22 February 2011. And the upper (green) curve includes all NZS4203 equivalent design demands cycles for a structure to survive the Christchurch Earthquake of 22 February 2011.

C = 1

In Figure 3, the upper graph is for the value of C = 1. This places a linear weighting on all amplitude cycles, such that two cycles at half the design amplitude equate to one design amplitude cycle. Thus the graph for C = 1, is used for determining the effective cyclic demands on components that possess a constant reserve of energy absorption capacity. This relates to damage to the components where the concrete is prone to failure (Dutta and Mander, 2001). In the context of the CTV Building, it is the connections that were prone to failure due to damage to the concrete. In particular the slab-to-precast beam connections and the beam-column joint connections are elements that are prone to concrete failure.

For the slab systems in the CTV Building, the vibration period is thought to be in the order of about 0.3 seconds. Thus from Figure 3, this implies from all earthquakes <u>prior to</u> the 22 February 2011 Christchurch earthquake some 85 NZS4203 code demand cycles would have been experienced by the structure. It is thus not surprising that the floor slabs felt quite lively by occupants, clear evident of repeated cyclic loading damage.

The beam column joint regions where the concrete was expected to be the sole mechanism to provide shear and bond resistance also experienced significant cyclic demand. Given the effective sway period would have been in the range of 1 second, it is evident that prior to the 22 February Christchurch Earthquake, some 20 cycles of loading would invariably cause damage to the beam-column joint concrete. Note that this demand exceeds the design expectation of 4-cycles by a wide margin.

For the CTV Building to survive the 22 February Christchurch Earthquake, there was a considerable additional cyclic demand. Given the effective period during this event would have shifted to 1.5 to 2 seconds, the total cyclic demand would be in the order of 30 cycles. With such demands it is not surprising the joint-zone concrete showed complete destruction, being pulverized to dust due to this repeated cyclic action.

C = 2

When C = 2, a cycle with 50% of the design amplitude is assumed to be provide fatigue demand equal to 25% of a full design amplitude cycle. This conforms to low cycle fatigue capacity of reinforcing steel (Mander el, 1994, and Dutta and Mander 2001). Reinforcing steel in plastic hinge zones is prone to premature low cycle fatigue failure. In the context of the CTV Building therefore, plastic hinges were observed to initiate in the structural walls.

Moreover, other buildings with more conventional ductile detailing are reinforcing steel-fatigue prone. It is therefore not surprising that other well designed and well detailed buildings have been condemned because of their uncertain remaining fatigue life. The ARCL designed IRD building would fall into this category where some 20 full design amplitude cycles of loading would have been experienced as a consequence of the earthquakes up to and including the 22 February Christchurch Earthquake.

In the case of the CTV Building a critical region where the reinforcing was prone to low cycle fatigue failure was the beam-column joints. This is because the concrete through the joints relative to the surrounding elements was relatively weaker leading to very high strain amplifications in the column joint steel. From Figure 3, it is indicated that some 20 cycles of NZS4203 code demand cycles would have been experienced by the critical reinforcing steel.

It should also be noted, that one cannot easily observe evidence of fatigue damage. Although cracking may be an indicator of a fatigue-prone location, the crack size cannot be used to infer the extent of fatigue damage. One must conduct some rational analysis using the principles of mechanics to understand the extent of fatigue damage.

C = 3

Finally, for sake of completeness, for C = 3, a cycle with amplitude equal to 50% of a design cycle will be deemed to contribute 12.5% of a design cycle damage. This category is applicable to fatigue damage in the steel components.

Implications of these results:

In accordance with NZS4203, if one assumes that there are to be 4 cycles at the design spectral displacement amplitude via the structure's capacity, then a capacity vs. demand evaluation can be made.

The CTV Building was exposed to cyclic demands considerably greater than what one would expect to observe back at the time structures were designed in the 1980's. Three implications arise:

1. Older buildings could not be expected to survive the cyclic demands exposed prior to and during the 22 February Christchurch Earthquake.
2. Given the forces that the building experienced in the 4 September 2010 earthquake, followed in close proximity by significant aftershocks, it would have been prudent for all concerned to have been suspicious about the ability of the CTV Building, designed as it was in 1986, to have with withstood the 4 September earthquake and immediate aftershocks without a material loss of fatigue capacity in fatigue-prone regions such as column bars and also its associated loss of strength in the concrete damage-prone elements, in particular the beam-column joints. Those suspicions could only be allayed by the performance of a structural analysis with references to the building plans, seismic and other information. A mere visual inspection would not be adequate.
3. Building survival to the excessive demands of the Canterbury earthquake sequence can only be attributed to a measure of over-strength that exists in structures where the in-situ strength exceeds the specified capacities by design. Ductility is not a substitute for strength, as a design concept it is shown to be wanting.

References:

Dutta, A. and Mander, J. B. (2001). "Energy based methodology for ductile design of concrete columns." ASCE *Journal of Structural Engineering* **127**(12): 1374-1381.

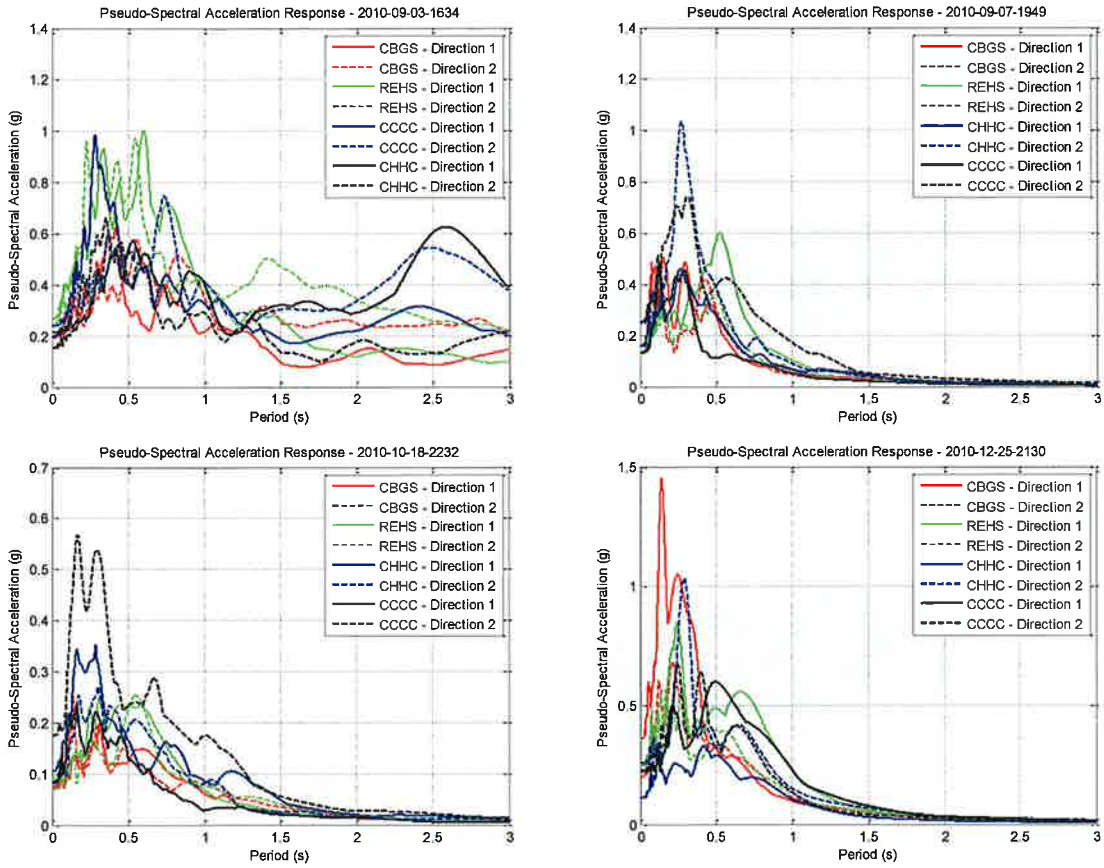
Dutta, A., Mander, J. B. and Kokorina, T. (1999). "Retrofit for control and repairability of damage." *Earthquake Spectra* **15**(4): 657-679.

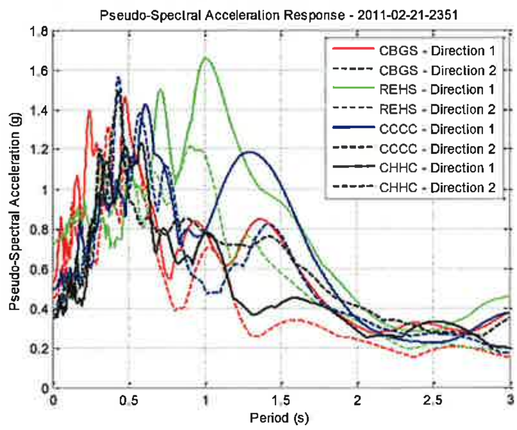
Mander, J., Panthaki, F. and Kasalanati, A. (1994). "Low-Cycle Fatigue Behavior of Reinforcing Steel." ASCE *Journal of Materials in Civil Engineering* **6**(4): 453-468.

Table 1: Major Christchurch earthquake events between September 4, 2010 and February 22, 2011.

					CBGS		CCCC		CHHC		REHS		CTV
	Event Time (Local Time)	Event Time (UTC)	Mag	Depth (km)	PGA (%g)	Distance (km)	PGA (%g)	Distance (km)	PGA (%g)	Distance (km)	PGA (%g)	Distance (km)	Distance (km)
3366146	Sat, Sep 4 2010 4:35 am	2010-09-03-1635	7.1	11	**18.86**	**36**	**23.81**	**39**	**21.38**	**37**	**26.28**	**38**	**38**
3366155	Sat, Sep 4 2010 4:56 am	2010-09-03-1656	5.3	8	0.29	32	-	34	-	32	1.01	33	33
3366230	Sat, Sep 4 2010 7:56 am	2010-09-03-1956	5.2	7	2.74	20	3.61	22	2.81	21	6.89	22	22
3366310	Sat, Sep 4 2010 11:12 am	2010-09-03-2312	5.3	12	0.81	28	1.21	31	-	29	-	30	30
3366313	Sat, Sep 4 2010 11:14 am	2010-09-03-2314	5.3	6	1.02	36	1.21	39	-	37	-	38	38
3366452	Sat, Sep 4 2010 4:55 pm	2010-09-04-0455	5.4	10	-	53	1.59	55	-	53	2.16	54	54
3367742	Mon, Sep 6 2010 11:24 pm	2010-09-06-1124	5.2	9	1.71	19	2.82	21	2.11	20	5.65	20	21
3367749	Mon, Sep 6 2010 11:40 pm	2010-09-06-1140	5.4	9	1.92	59	4.73	61	3.17	60	7.40	61	61
3367832	Tue, Sep 7 2010 3:24 am	2010-09-06-1524	5.4	15	1.85	33	3.73	35	2.81	34	6.28	35	35
3368445	Wed, Sep 8 2010 7:49 am	2010-09-07-1949	5.1	6	15.79	7.9	25.40	5.8	24.57	7.0	13.53	7.7	6.5
3382676	Mon, Oct 4 2010 10:21 pm	2010-10-04-0921	5.0	12	1.64	13	2.91	15	1.84	14	4.27	15	15
3388384	Wed, Oct 13 2010 4:42 pm	2010-10-13-0342	5.0	15	1.10	16	1.99	18	1.43	17	3.34	18	18
3391440	Tue, Oct 19 2010 11:32 am	2010-10-18-2232	5.0	9	7.30	12	17.51	12	10.43	12	9.04	13	13
	Sun, Dec 26 2010 10:30 am	2010-12-25-2130	4.9	5	35.54	2.1	22.36	1.7	25.49	1.4	24.40	2.7	1.9
3450113	Thu, Jan 20 2011 6:03 am	2011-01-19-1903	5.1	10	2.10	11	5.28	12	3.35	11	3.38	13	12
3468575	Tue, Feb 22 2011 12:51 pm	2011-02-21-2351	6.3	5	53.06	11	49.31	8.5	36.73	9.8	71.82	10.4	9.3

These summary results are obtained by loading, processing and filtering the raw recording station outputs for the events below as most of these events are not available as processed records on the GeoNet ftp site. Absent fields indicate that no recording occurred at that station for that event. PGA values represent the peak horizontal acceleration from either orthogonal direction (%g)





Notes:

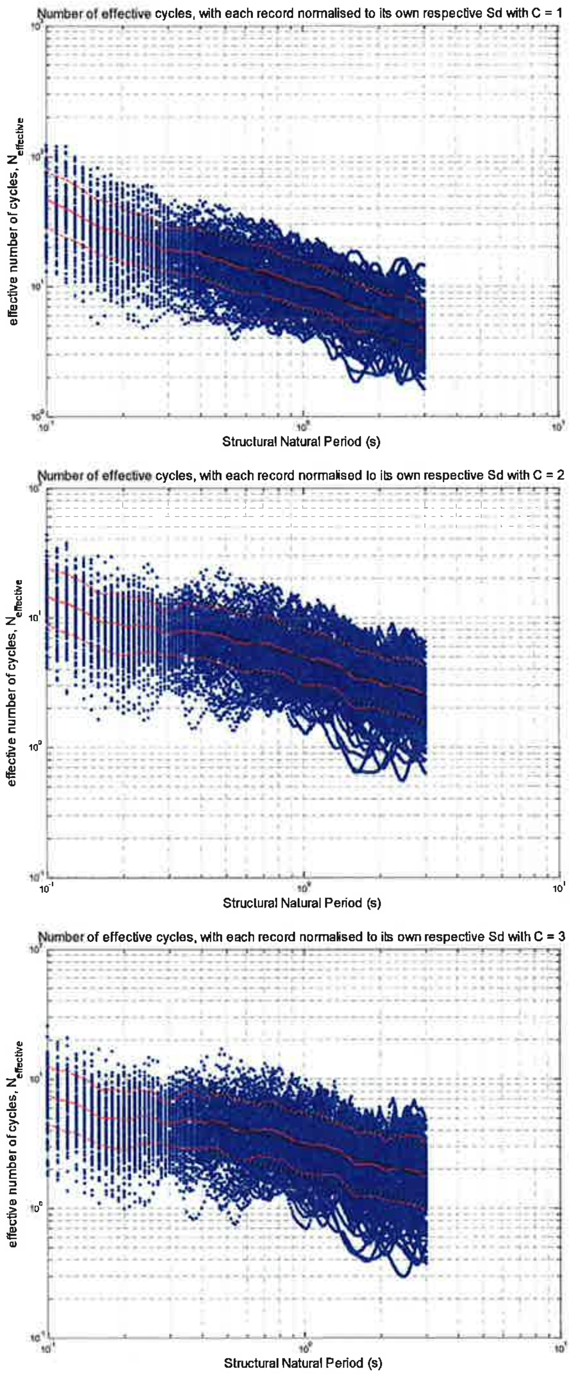
1. These response spectra are presented in chronological order for the response of structures with standard 5% inherent damping.
2. Two lines are presented for each station which corresponds to the two orthogonal horizontal directions.
3. The periods modelled are 0.001 to 3.0s natural periods, in increments of 0.001s.
4. The spectral response at T = 0s period represents the PGA for that ground motion input.
5. The spectra presented here relate to the five events with the highest PGA at the four CBD stations of interest (CBGS, CCCC, CHHC and REHS) in the time period from 4 September 2010 to 22 February 2011 inclusive.

Figure 1. Pseudo-Spectral Acceleration Spectra for the major events between September 4, 2010 and February 22, 2011

C = 1

C = 2

C = 3



Notes:

a. Results are for each orthogonal direction for each station and all 16 major events – 122 ground motion records

b. The equivalent amplitude is defined as the spectral displacement, Sd, for that particular record.

c. The upper, central and lower red lines show the 84^{th}, 50^{th} and 16^{th} percentiles for a log-normal distribution. The 50^{th} percentile is the median result, while the 84^{th} and 16^{th} percentile are ± one log-normal standard deviation.

d. The black lines represent the fitted equations to the three red lines, with the equations show on the graph.

e. The variability of the results for any given period is due to the record-to-record randomness in the ground motion amplitude and duration input. This leads to a random fatigue demand with a measured dispersion (based on the underlying lognormal distribution) of β.

f. β has a value of 0.41, 0.48 and 0.54 for C = 1, 2, and 3 respectively.

1. For Concrete-critical fatigue, C=1
$N_{eff[C=1]} = 10.1\ T^{-0.667}$

2. Reinforcing-steel fatigue, C = 2
$N_{eff[C=2]} = 4.91\ T^{-0.6}$

3. Structural steel with fracture-critical details C = 3
$N_{eff[C=3]} = 3.11\ T^{-0.5}$

Figure 2: Equivalent cycles, $N_{effective}$ vs. Period for all records

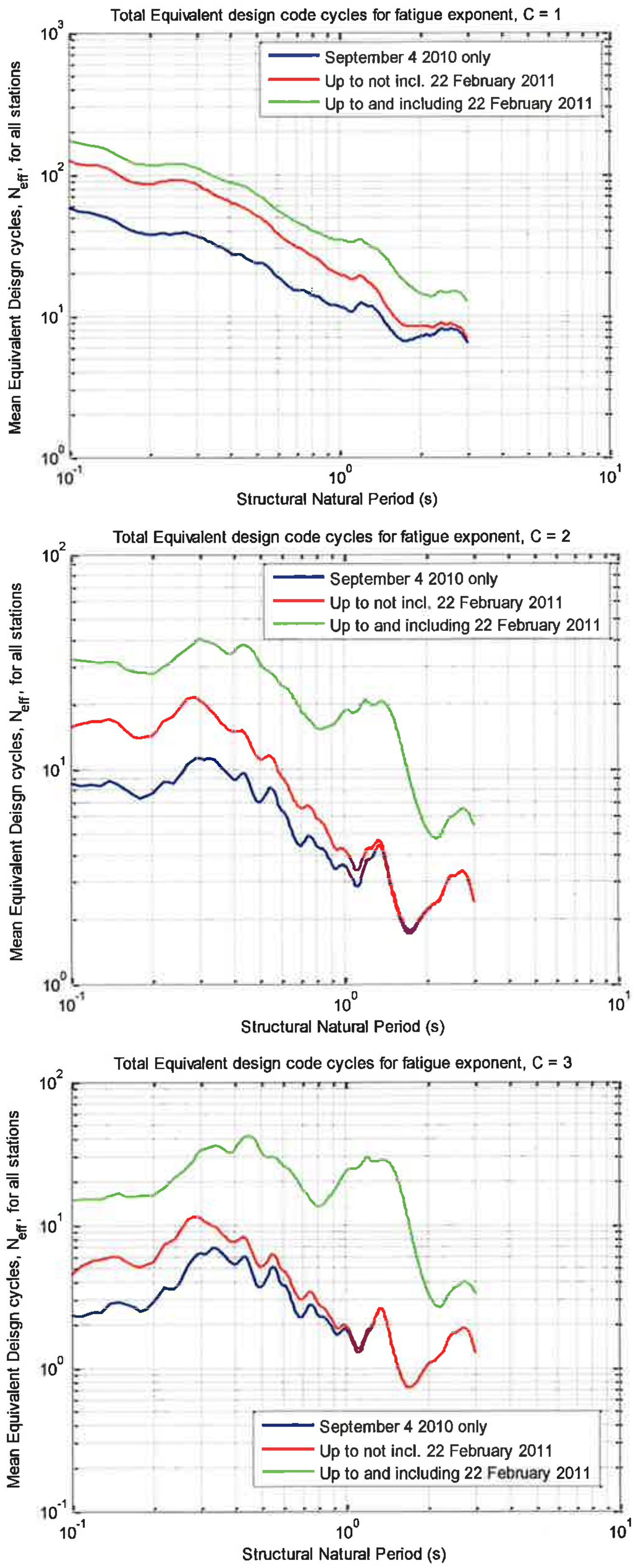


Figure 3: Fatigue demand spectra for major earthquakes from 4 September 2010 to 22 February 2011.

Notes:

a. The number of equivalent fatigue cycles with amplitudes equal to that defined in NZS4203-84 design loadings code implied for an elastic structure.

b. Results are calculated at each period point for each of the three fatigue exponents C = 1, 2, and 3.

c. The lower blue curves plot the computed results for the September 4, 2010 main shock only

d. The middle red curves plot the results for all earthquakes up to, but not including the February 22, 2011 main shock.

e. The upper green curves plot the total number of equivalent number of equal-amplitude fatigue demand cycles every major event up to and including the February 22, 2011 earthquake.

f. In the fatigue demand analysis the fatigue exponents of C = 1, 2 and 3 are used as they are typical of the experimentally observed fatigue capacity of for:

4. Concrete-critical fatigue, C=1.

5. Reinforcing-steel fatigue, C = 2

6. Structural steel with fracture-critical details C = 3