SECTION 2

PROGRESSIVE DAMAGE ACCUMULATION IN EARTHQUAKES

WITHIN THE CONTEXT OF THE CTV BUILDING COLLAPSE.

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Contextual Background

Two key factors that lead to the overall damage effects on structures:

1. *Maximum response* displacement or drift

2. *Duration* of the earthquake, which leads to ***cyclic loading effects***

Cyclic Loading Demands Lead to FATIGUE Failures

1. High-cycle fatigue:

- Aircraft wing flutter
- Engine vibrations
- Bridge deck vibrations
- Material behavior must remain elastic.
- > 2 million cycles: total stress range < 150 MPa.

2. Low-cycle fatigue:

- Material behavior: inelastic (post-yield)
- Fracture can occur in reinforcing bars during earthquakes
- Can lead to deterioration of concrete
- Phenomena well researched, but not well applied in practice.

Is Strain-Hardening a surrogate for Low Cycle Fatigue?

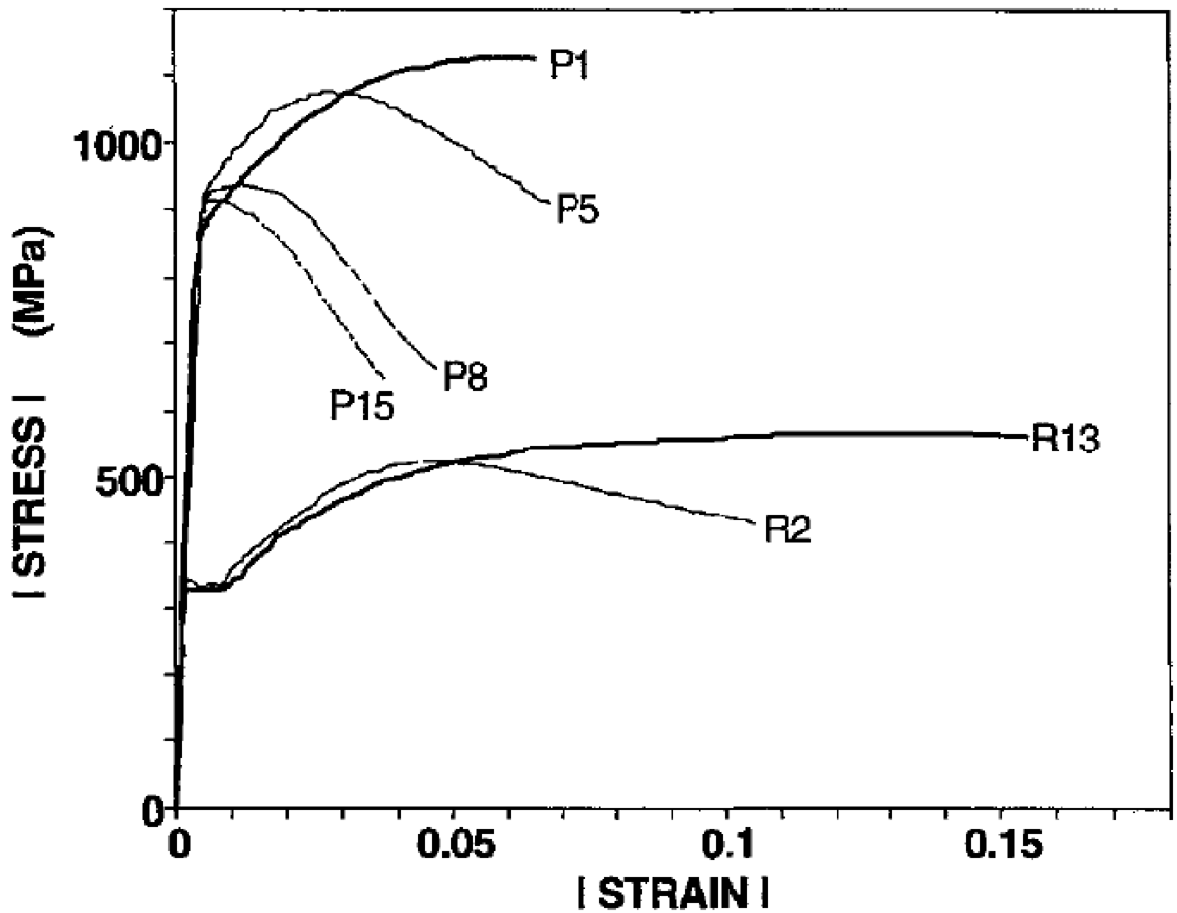
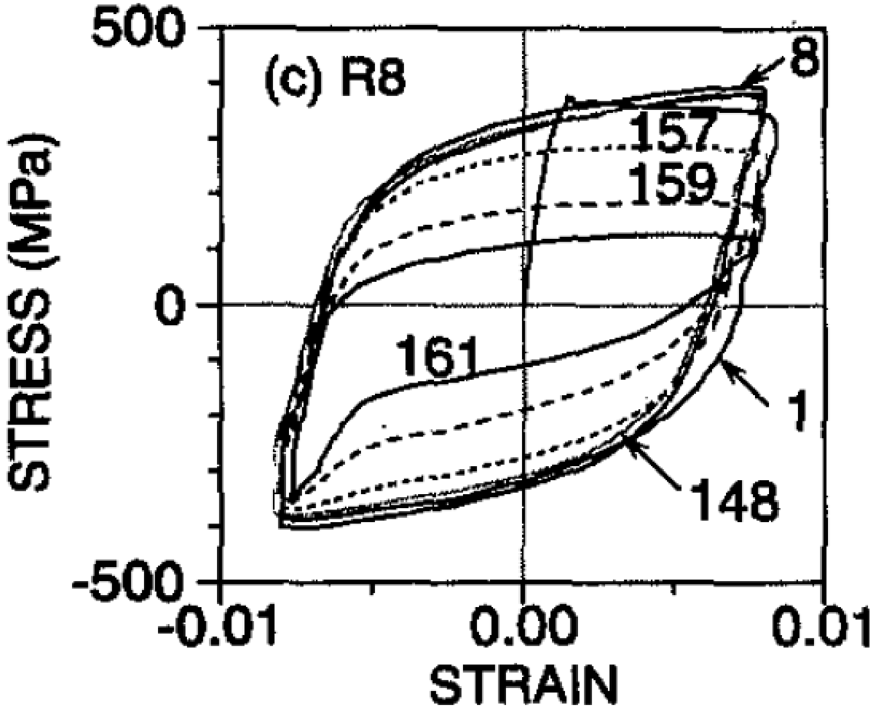


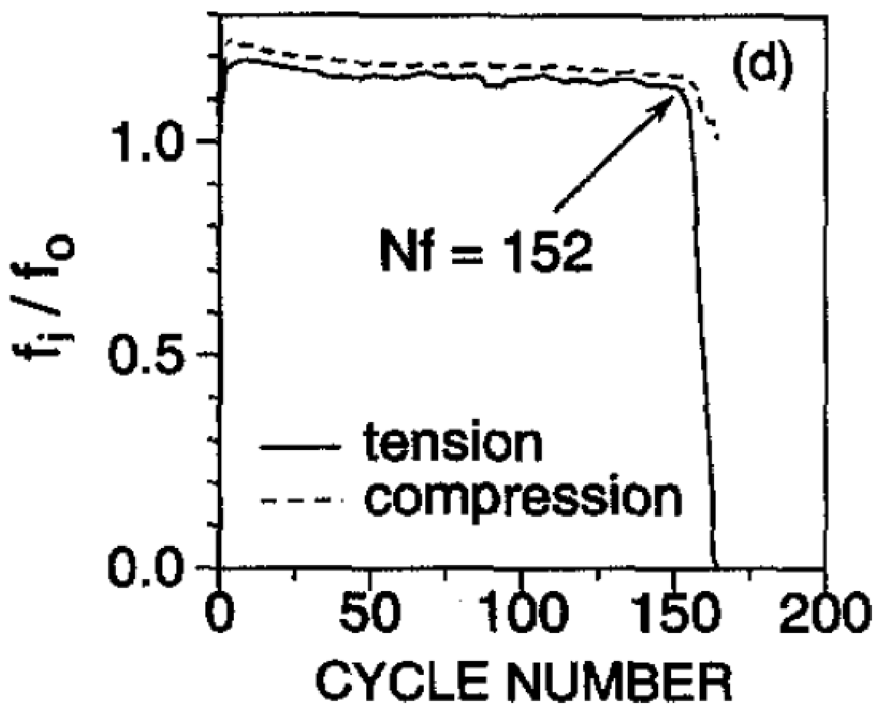
FIG. 2. Stress-Strain Results for Monotonic Tests

Mander et al., (1994) "Low-cycle fatigue of reinforcing steel,"

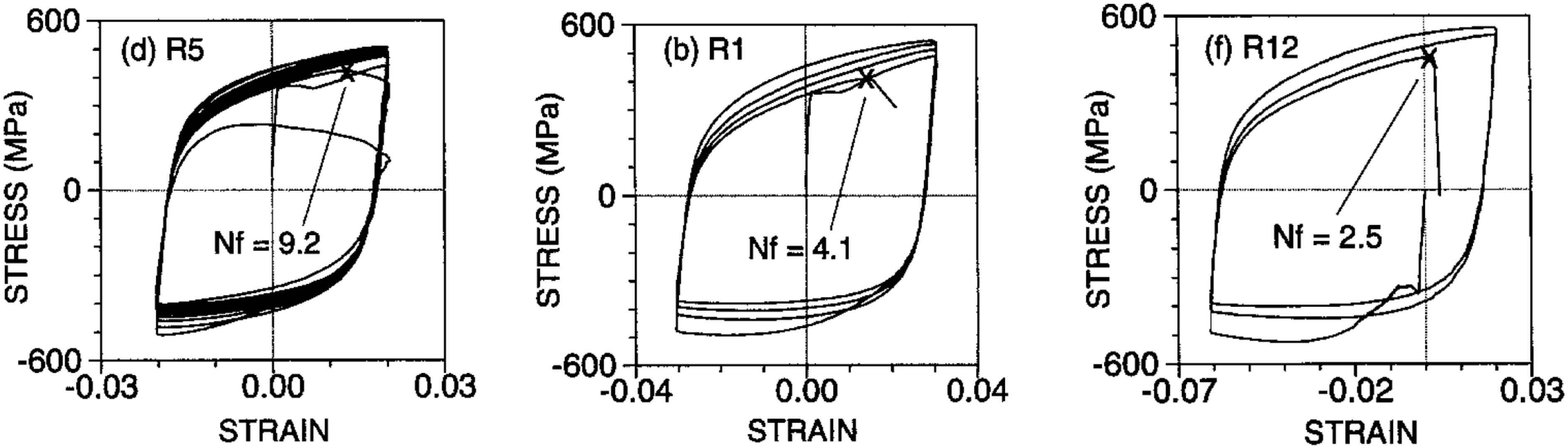
Journal of Materials in Civil Engineering, ASCE , Vol. 6, No. 4, pp. 453-468

Cyclic loading without strain-hardening

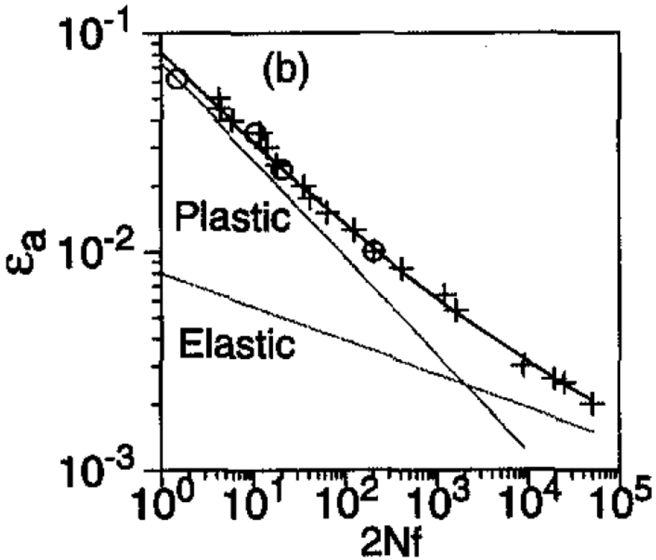




Cyclic loading <u>with</u> strain-hardening

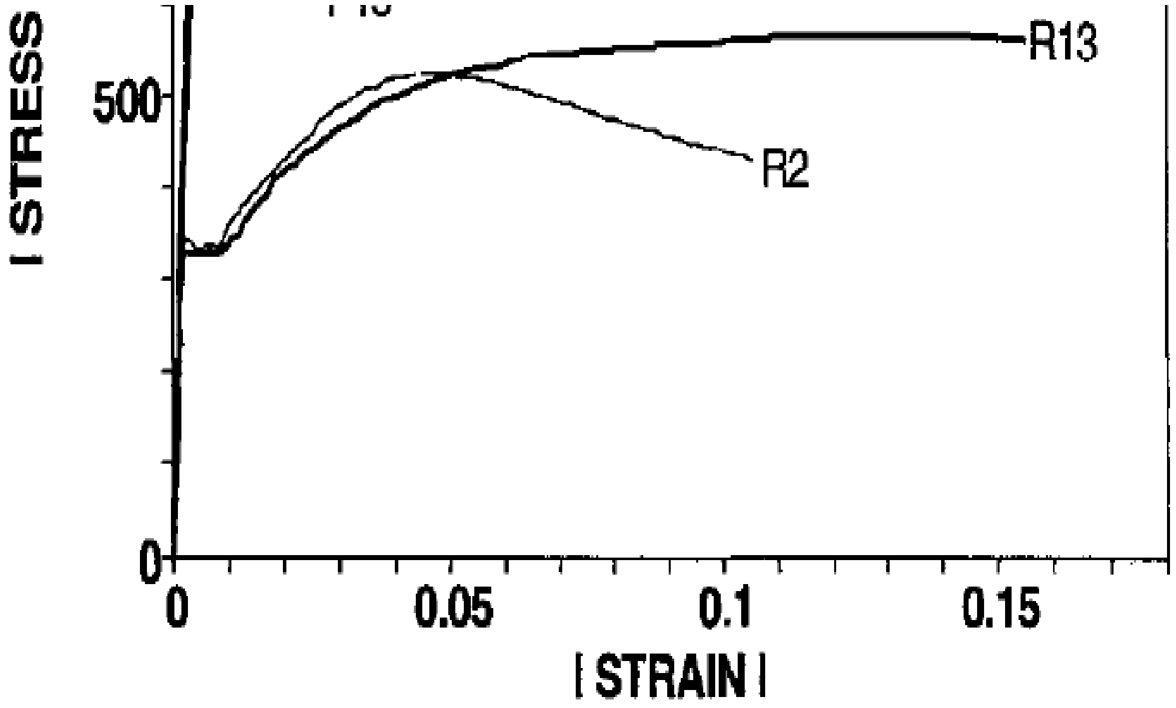


$\varepsilon_{ap} =$ 0.018 0.027 0.037



$$\varepsilon_{ap} = 0.08(2N_f)^{-0.5}$$

Q: *Is Strain-Hardening a good surrogate for Low Cycle Fatigue?*



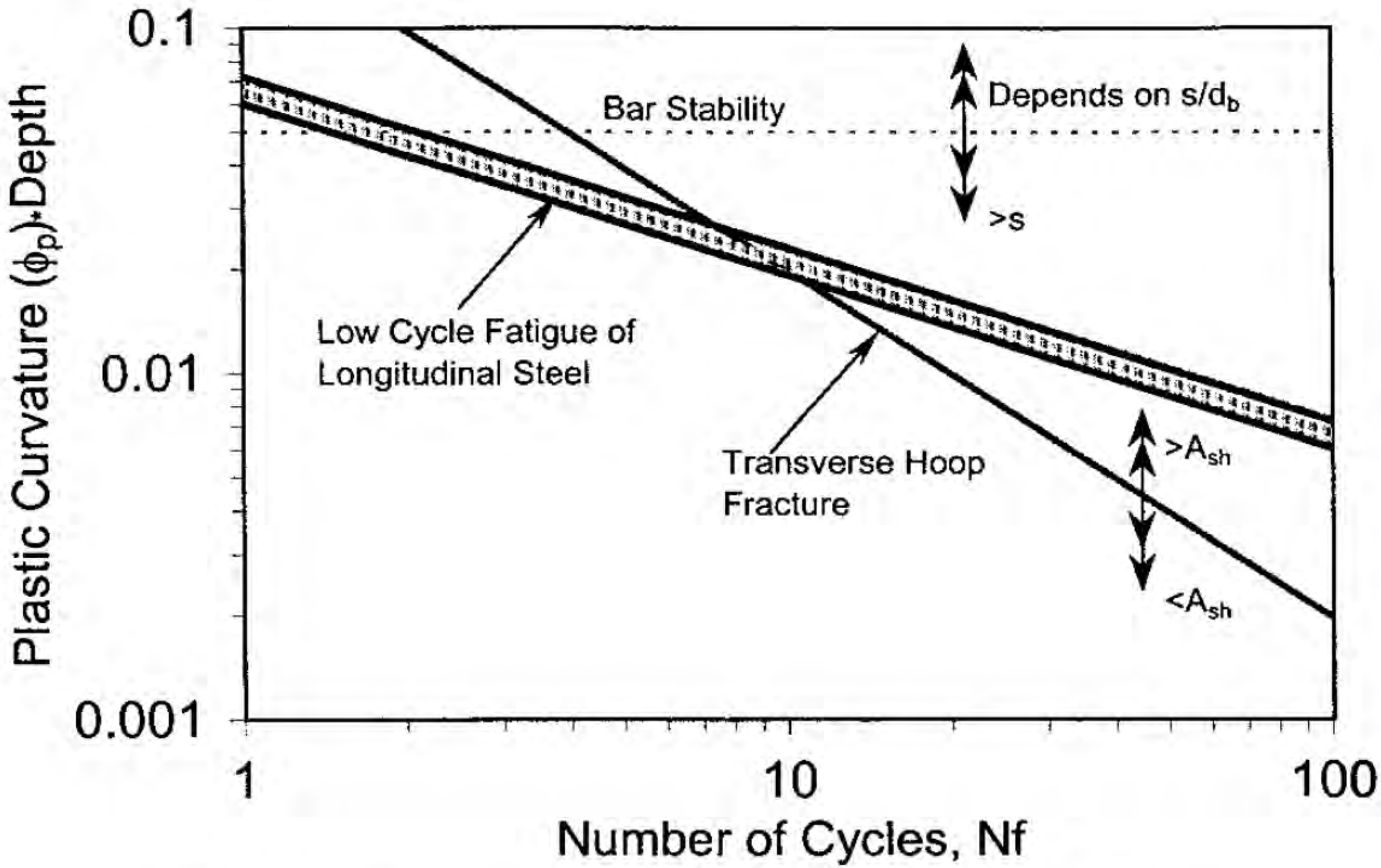
A: *possibly yes, but the phenomena should be referred to as LCF.*

RC Fatigue Capacity (Dutta and Mander, 2001)

- 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.
- General fatigue capacity: $$\phi_p D = \Theta'_{\text{hoop}}(2N_c)^c \quad (1)$$
- Concrete fatigue: $$\phi_p D = \Theta^{\text{circ}}_{\text{hoop}}(2N_c)^{-1} \quad (12)$$
- Steel (plastic) LCF $$\varepsilon_{ap} = 0.08(2N_f)^{-0.5} \quad (24)$$
- RC fatigue $$\phi_p D = 0.16\,\frac{D'}{D}\,(2N_f)^{-0.5} \quad (27)$$
- Inverting $$N_{cycles} = aX^C$$

RC Fatigue Capacity (Dutta and Mander, 2001)

$$N_{cycles} = aX^{C}$$



Modification of fatigue theory to accommodate high axial load fluctuation

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

- Normal fatigue capacity for constant axial load

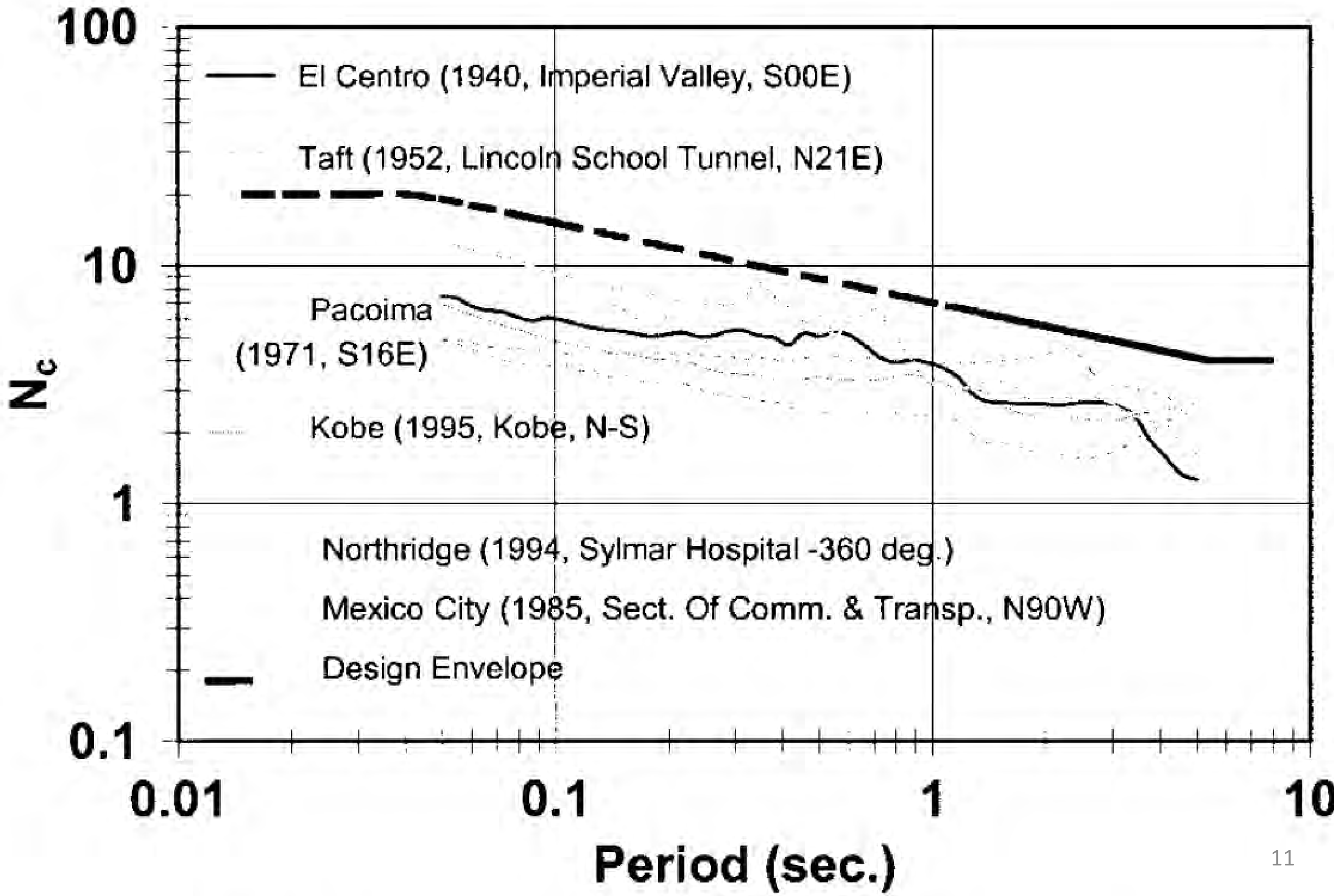
$$N_f^{theory} = \frac{0.013}{\theta_p^2}\left(\frac{L_f}{D'}\right)^2 \tag{4}$$

- Modification for change in axial load effects:

$$N_f^* = \frac{N_f}{\left(1 + 1.73\dfrac{\Delta P}{f_c^{'} A_g}\right)^3} \tag{13}$$

- where ΔP = change in axial load due to either framing effects or vertical acceleration induced response.
- For CTV lower story: $N^*_f = 0.33\ N_f$

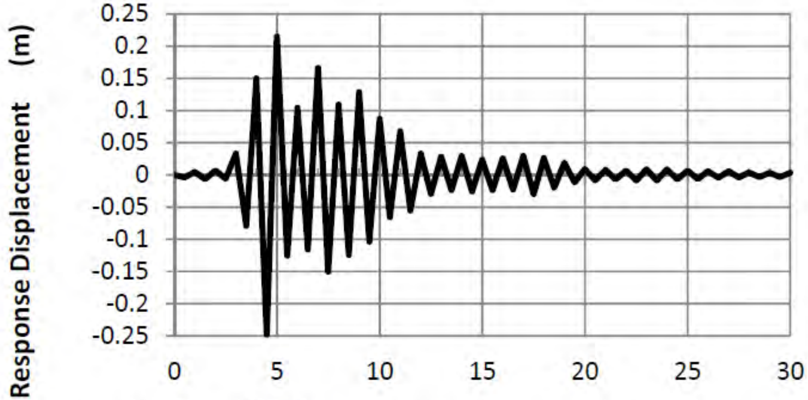
Seismic Fatigue Demand



The Cyclic Loading Demands of Earthquakes

- Qualitatively appreciated since the 1970's
- Not well quantified due to lack of ground motion records prior to 1989.
- Clause C3.2 of NZS3101 states:
  - The structure should be capable of sustaining four fully reversed cycles of loading without loosing more than 20% of its strength capacity.
  - Is this a realistic measure?

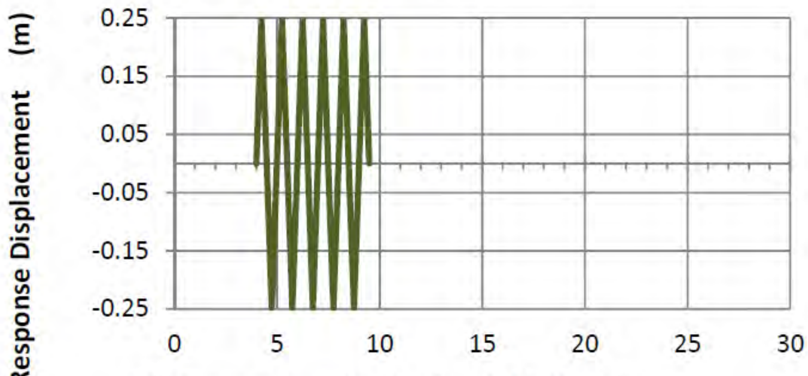
Fatigue Demand



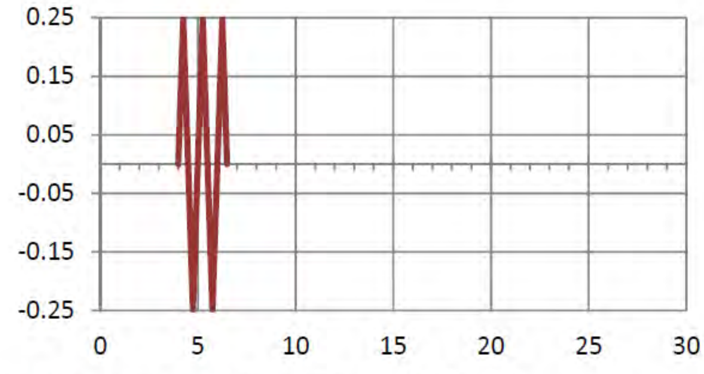
(a) Kobe Earthquake motion

Notes:

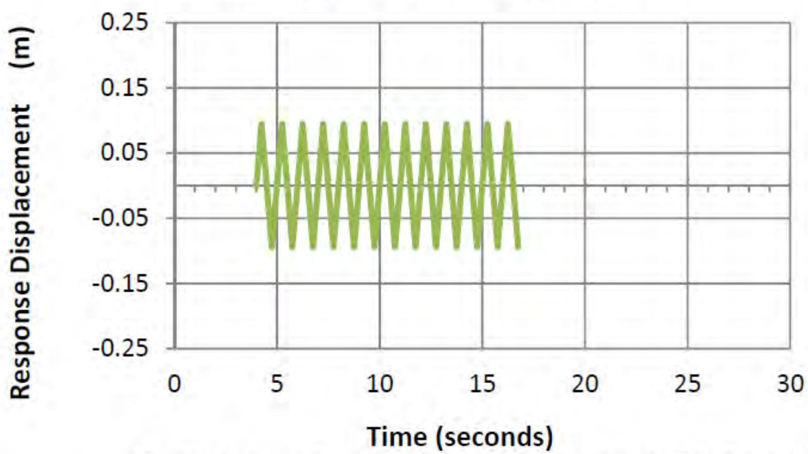
- C = 1 represents the case of constant energy capacity for the material such as the plain concrete and confined concrete where hoop fracture may occur
- C = 2 represents the case for low cycle fatigue of reinforcing steel
- For graphs (d) and (e) the design spectral displacement in this example is 94.4 mm.



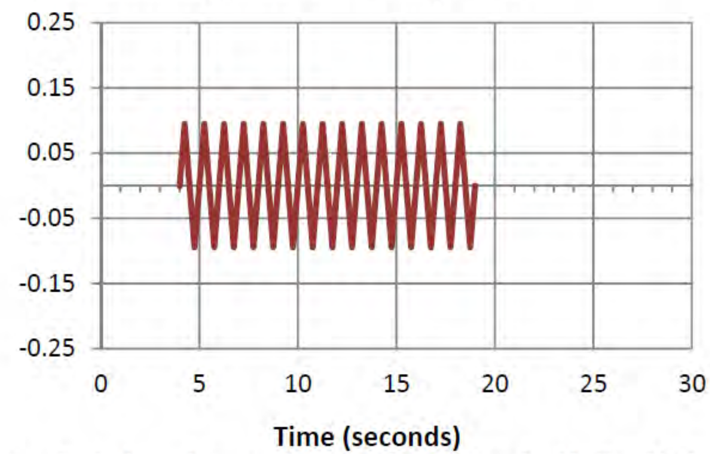
(b) Constant amplitude, C = 1, Neff = 5.4



(c) Constant amplitude, C = 2, Neff = 2.5



(d) Code-based equivalent amplitude C = 1, Neff = 14.2



(e) Code-based equivalent amplitude C = 2, Neff = 17.1

Fatigue - Equivalent Cycles

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

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

where x_i is the i^{th} displacement points and C is the fatigue exponent. The mean, m, of all displacement point can be determined from:

$$m = \frac{\sum_{i=1}^{N} \left(\frac{|x_i|}{A_{ref}} \right)^C}{n_{points}}$$

Where n_{points} is the total number of data points for that record, ie: $n_{points} = t_f/dt$, where t_f is the final time for the record, and dt is the time step. 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 = 1.414. For C = 1, B = 1.57 and for C = 3, B = 1.33. Therefore the effective amplitude becomes:

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

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

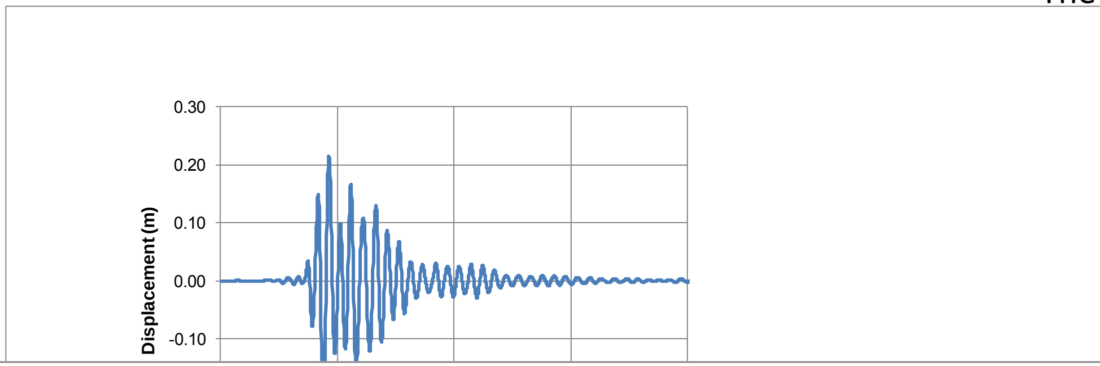
$$N_{cycles} = \frac{n_{points}\ \Delta t}{T} = \frac{t_f}{T}$$

where T is the natural period of the structure of interest. Finally, the number of effective cycles at the reference amplitude can be determined from:

$$N_{eff} = N_{cycles} \left(A_{eff} \right)^C$$

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

Fatigue – Area Equivalence



- The area underneath the
acement-time response
ained from a linear time-history
ysis) is integrated.
integrated area is then compared
the area enclosed within one
-reversed sine-wave at the design
od and amplitude of interest, A_{ref}.

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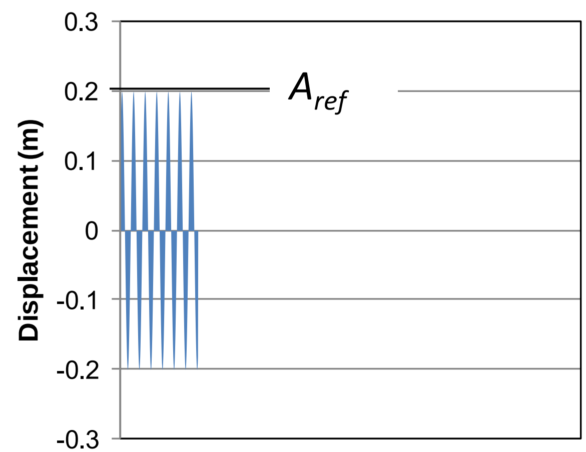


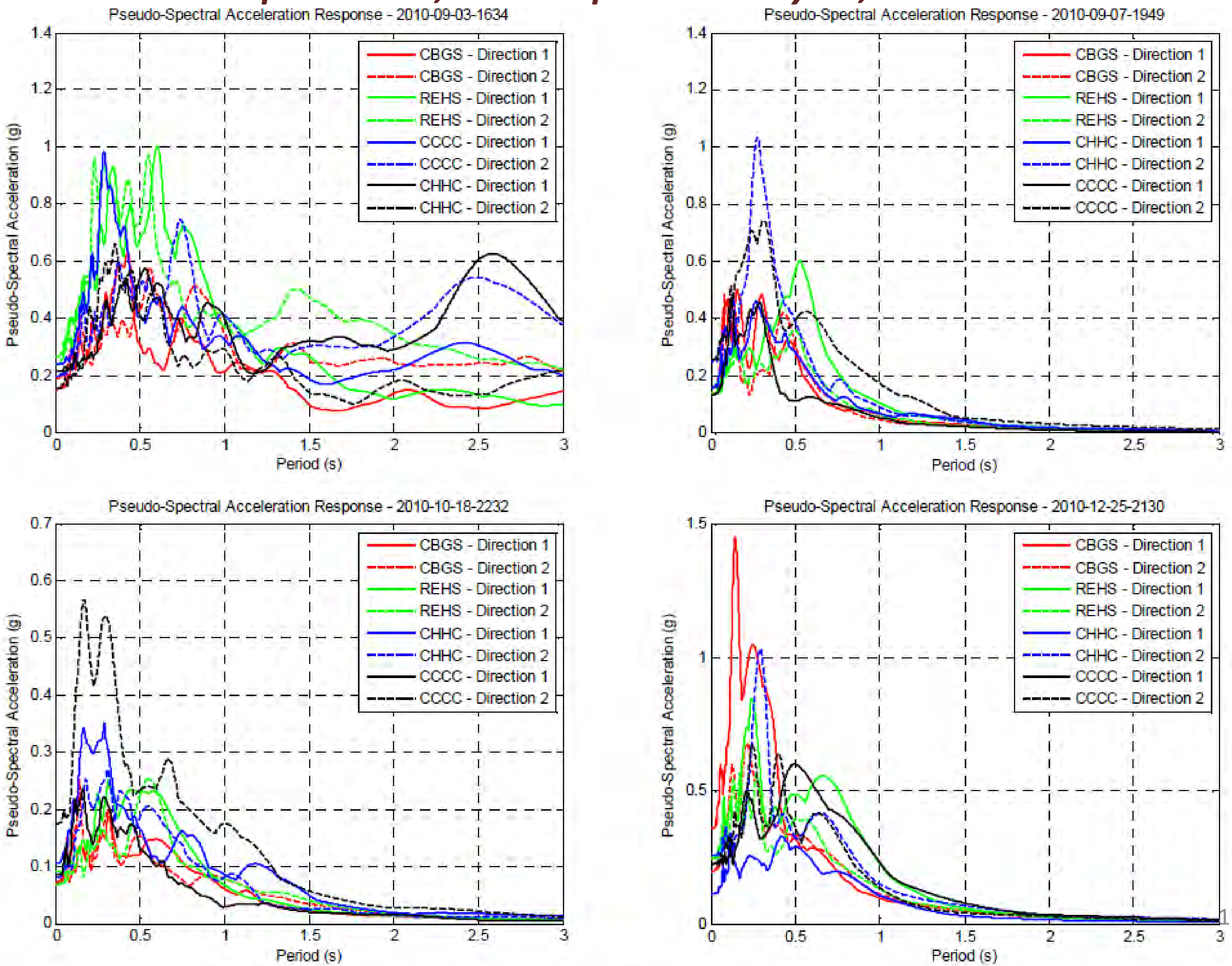
Equate the enclosed area into equivalent cycles at a reference amplitude.

TABLE 1: MAJOR CHRISTCHURCH EARTHQUAKE EVENTS BETWEEN SEPTEMBER 4, 2010 AND FEBRUARY 22, 2011

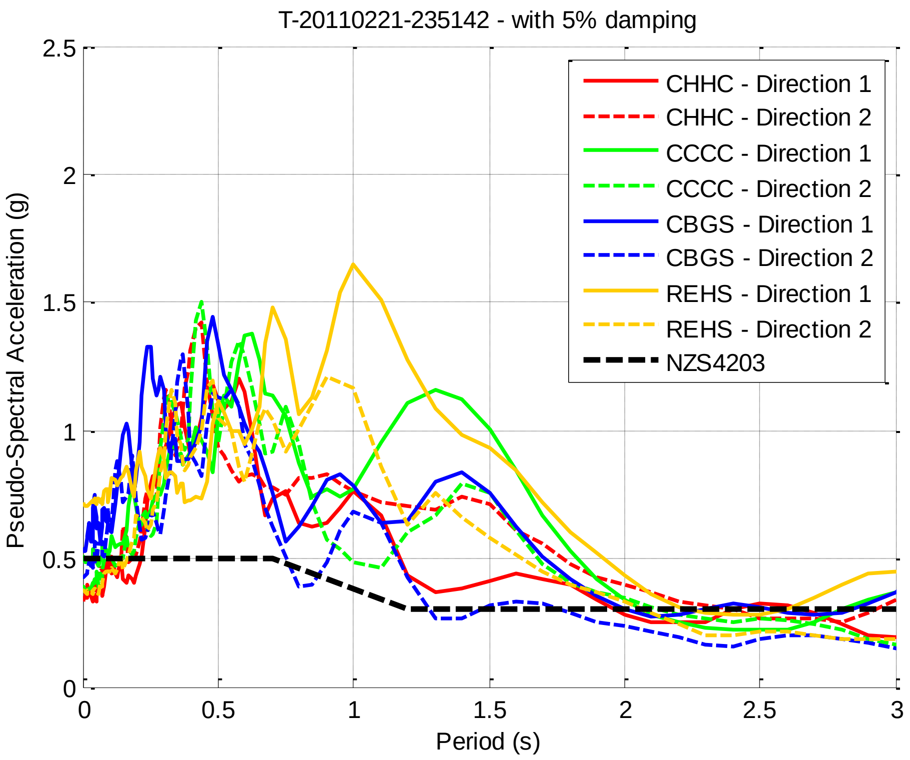
					CBGS		CCCC		CHHC		REHS		CTV
	Event Time (Local Time)	Event Time (UTC)	Mag	Depth (km)	PGA (%g)	Distance (km)	PGA (%g)	Distance (km)	PGA (%g)	Distance (km)	PGA (%g)	Distance (km)	Distance (km)
3366146	Sat, Sep 4 2010 4:35 am	2010-09-03-1635	7.1	11	**18.86**	**36**	**23.81**	**39**	**21.38**	**37**	**26.28**	**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

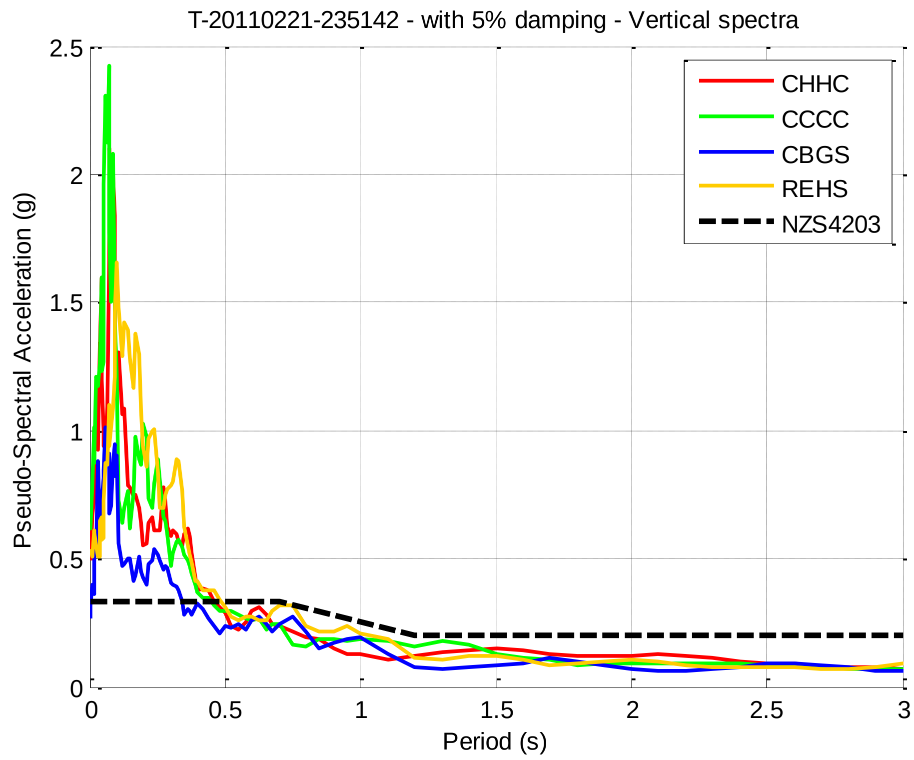
These summary results are obtained by loading, processing and filtering the raw recording station outputs for the events below as most of these events are not available as processed records on the GeoNet ftp site. Absent fields indicate that no recording occurred at that station for that event. PGA values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

Figure 1. Pseudo-Spectral Acceleration Spectra for the major events from September 4, 2010 to pre February 22, 2011

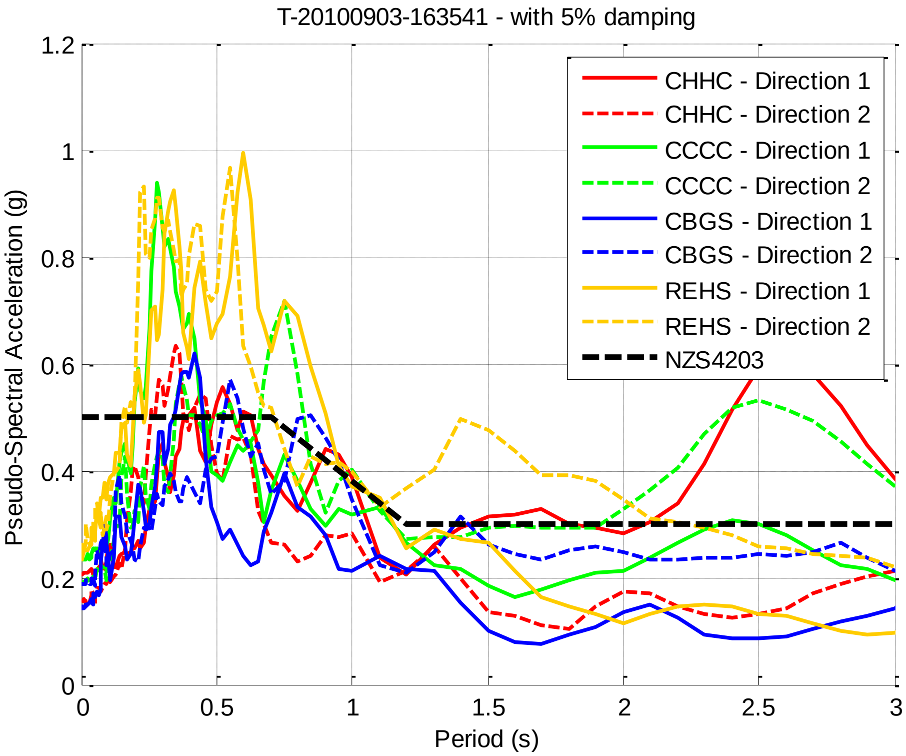


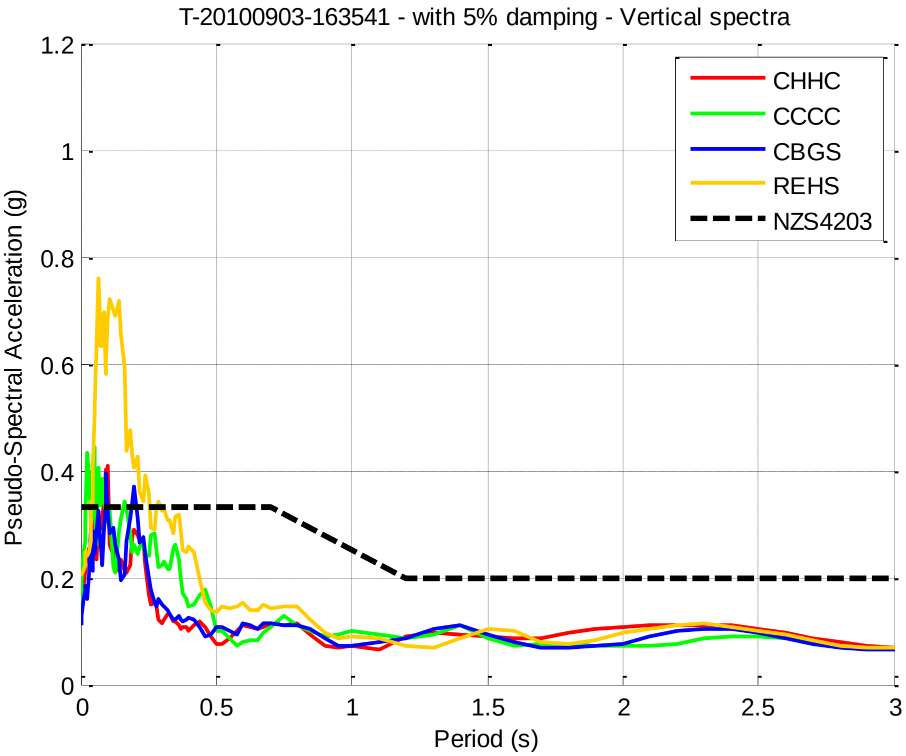
Christchurch Earthquake 22/2/11 Response Spectra



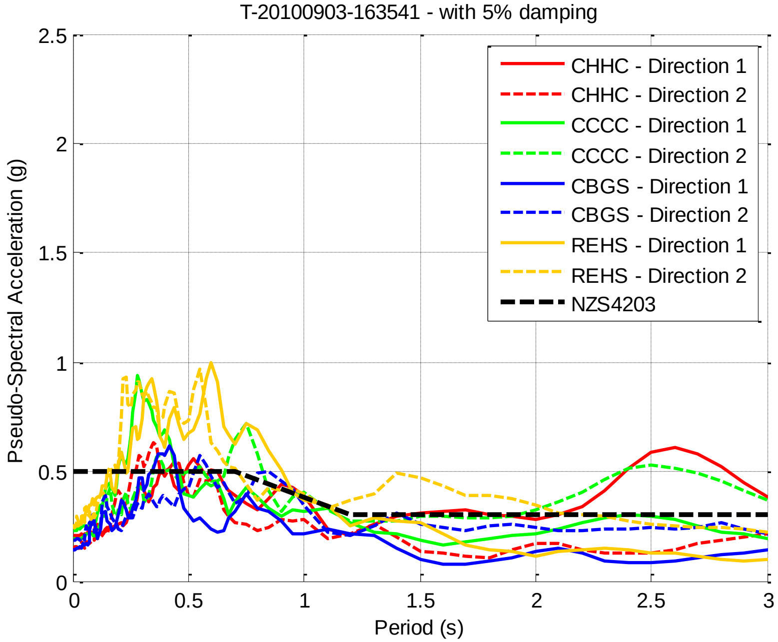


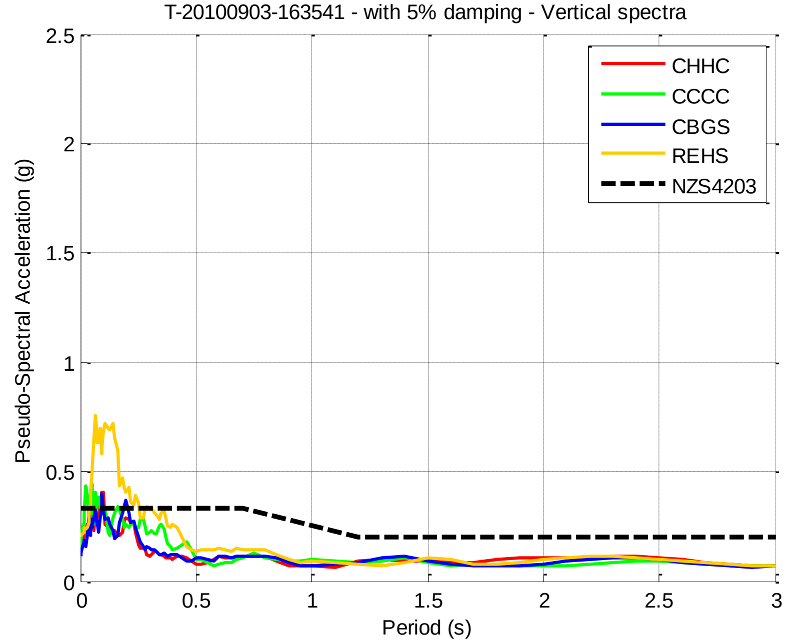
Darfield Earthquake 4/9/10 Response Spectra

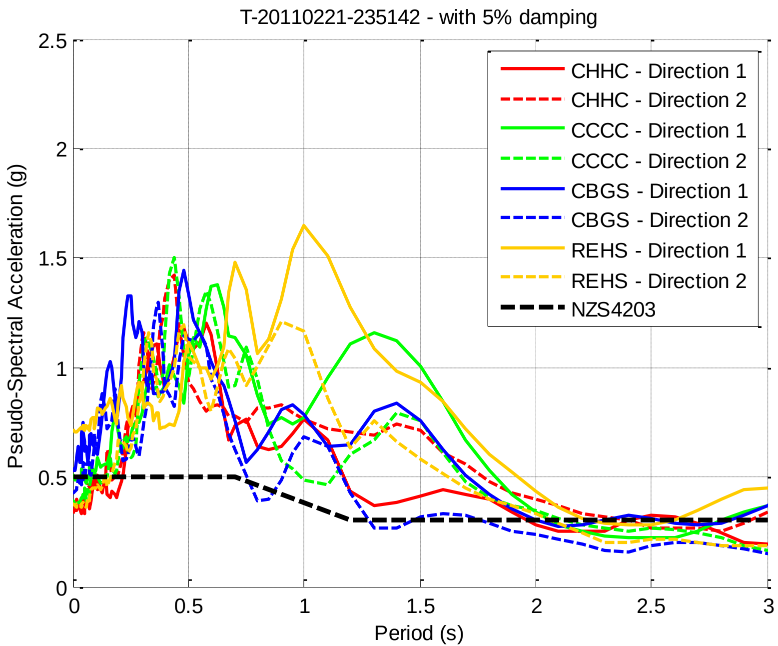


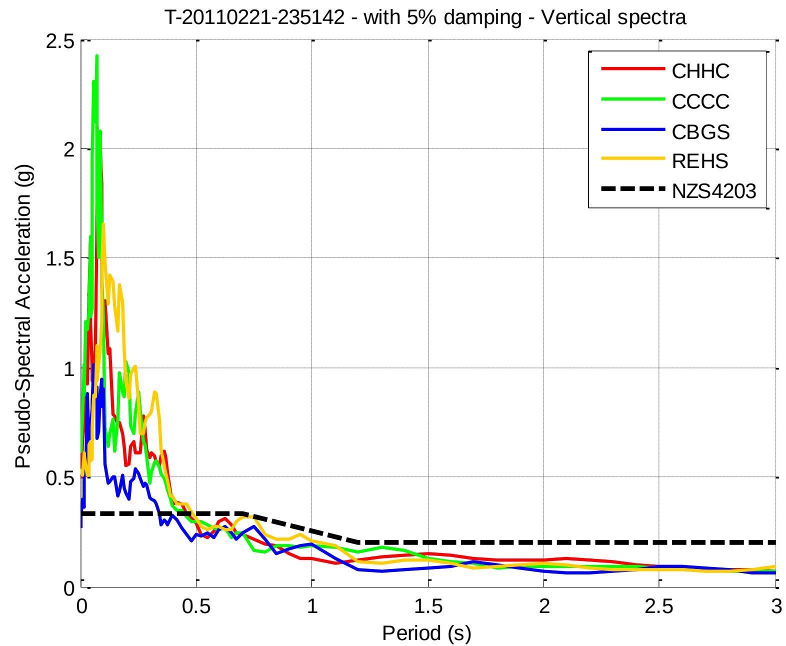


Comparison


T-20100903-163541 - with 5% damping
Pseudo-Spectral Acceleration (g)
Period (s)
CHHC - Direction 1
CHHC - Direction 2
CCCC - Direction 1
CCCC - Direction 2
CBGS - Direction 1
CBGS - Direction 2
REHS - Direction 1
REHS - Direction 2
NZS4203


T-20100903-163541 - with 5% damping - Vertical spectra
Pseudo-Spectral Acceleration (g)
Period (s)
CHHC
CCCC
CBGS
REHS
NZS4203


T-20110221-235142 - with 5% damping
Pseudo-Spectral Acceleration (g)
Period (s)
CHHC - Direction 1
CHHC - Direction 2
CCCC - Direction 1
CCCC - Direction 2
CBGS - Direction 1
CBGS - Direction 2
REHS - Direction 1
REHS - Direction 2
NZS4203


T-20110221-235142 - with 5% damping - Vertical spectra
Pseudo-Spectral Acceleration (g)
Period (s)
CHHC
CCCC
CBGS
REHS
NZS4203

Next few slides:

- A review of recent evidence from Heyward and Frost focusing on the beam-column and beam-slab failures.
- Hypothesis: Very high vertical motions (> 1g) left the slab-beam connections “broken” (my coined phase), or in Canterbury terms “they’re munted.”
- Note, the slabs and the vertical motion in general had a higher vibration frequency, T~0.25 sec.

Figure 47

Section through precast internal beam to slab (extract form Dwg S 15). The typical failure surface observed is indicated.

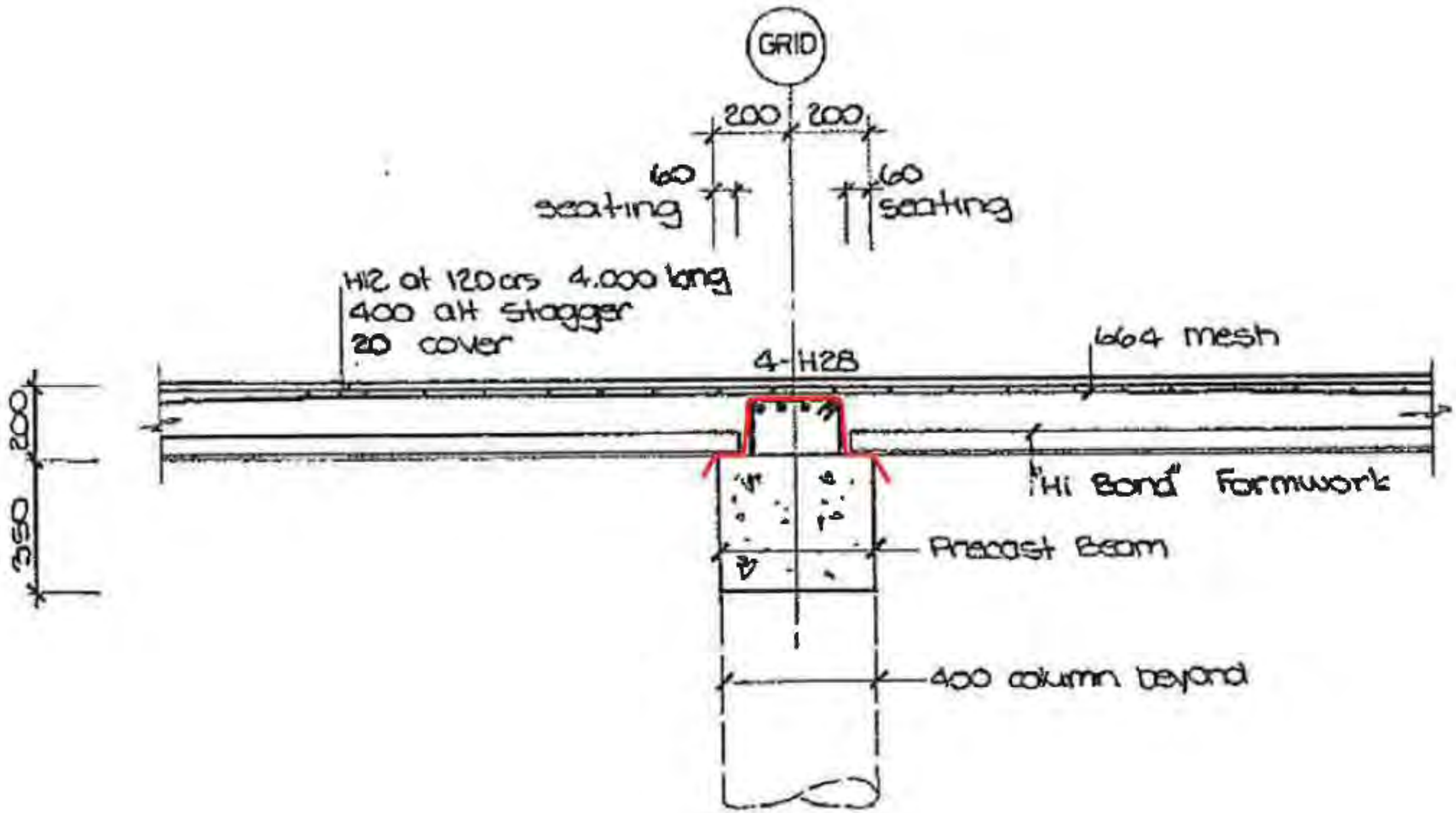


Figure 49

A precast internal beam lying on its side after extraction from the Building. The beam had completely separated from the slab during the collapse as indicated by the fire blackened concrete on the failure surface. The bent bars protruding from the end, the cylindrical surface on the end of the beam and the oxy-cut top reinforcing bars indicate that the end of the beam to the left of the photograph was supported on a round internal column.

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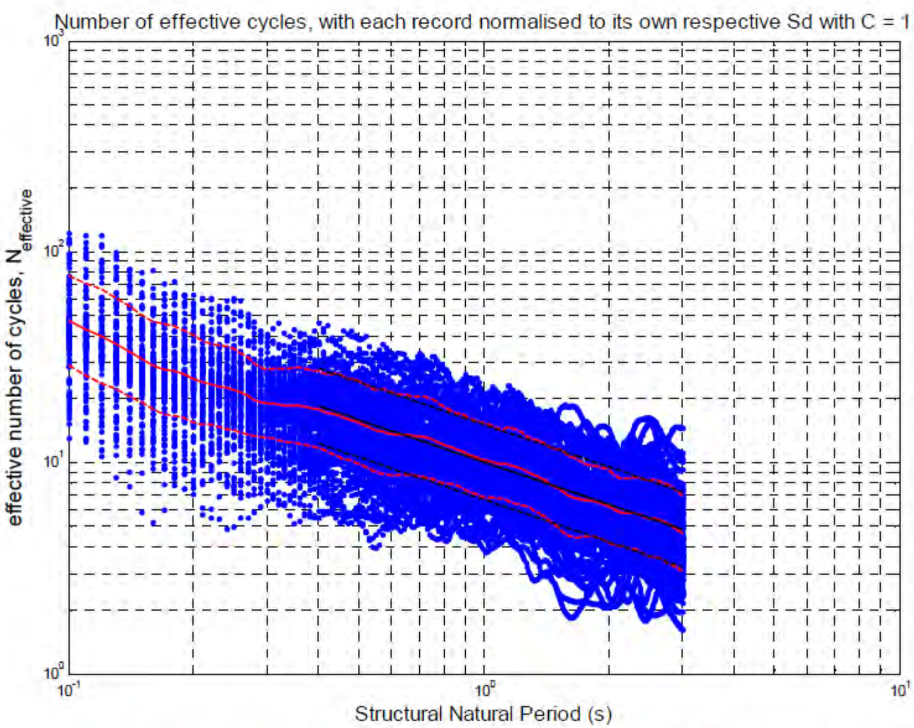
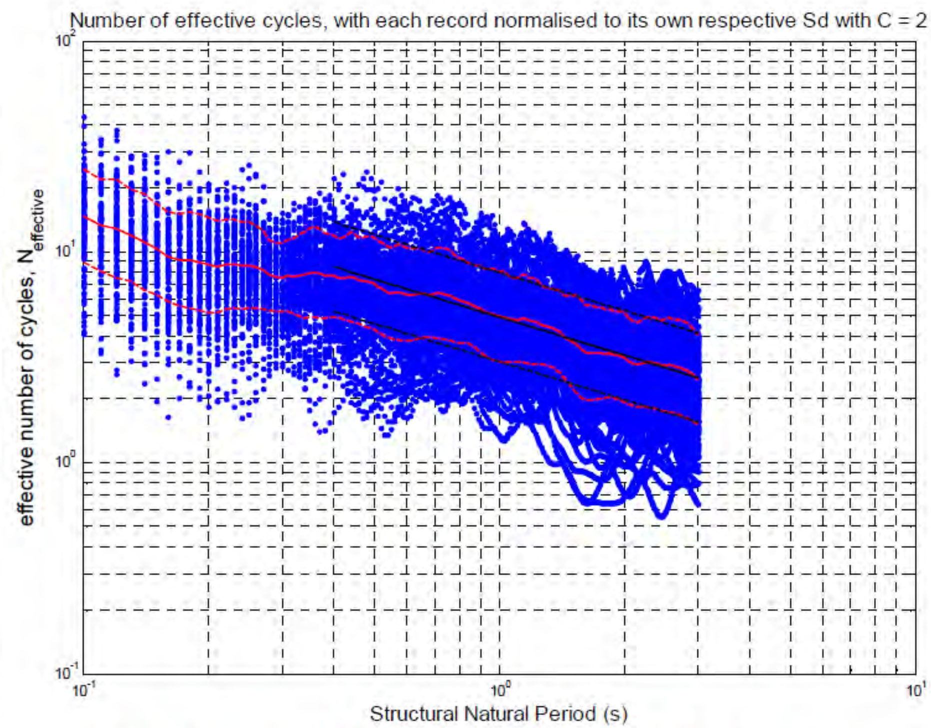
Figure 2: Equivalent cycles: $N_{effective}$ vs. Period *(for all records)*

Note: Cycle counting is normed back to the spectral displacement, Sd.

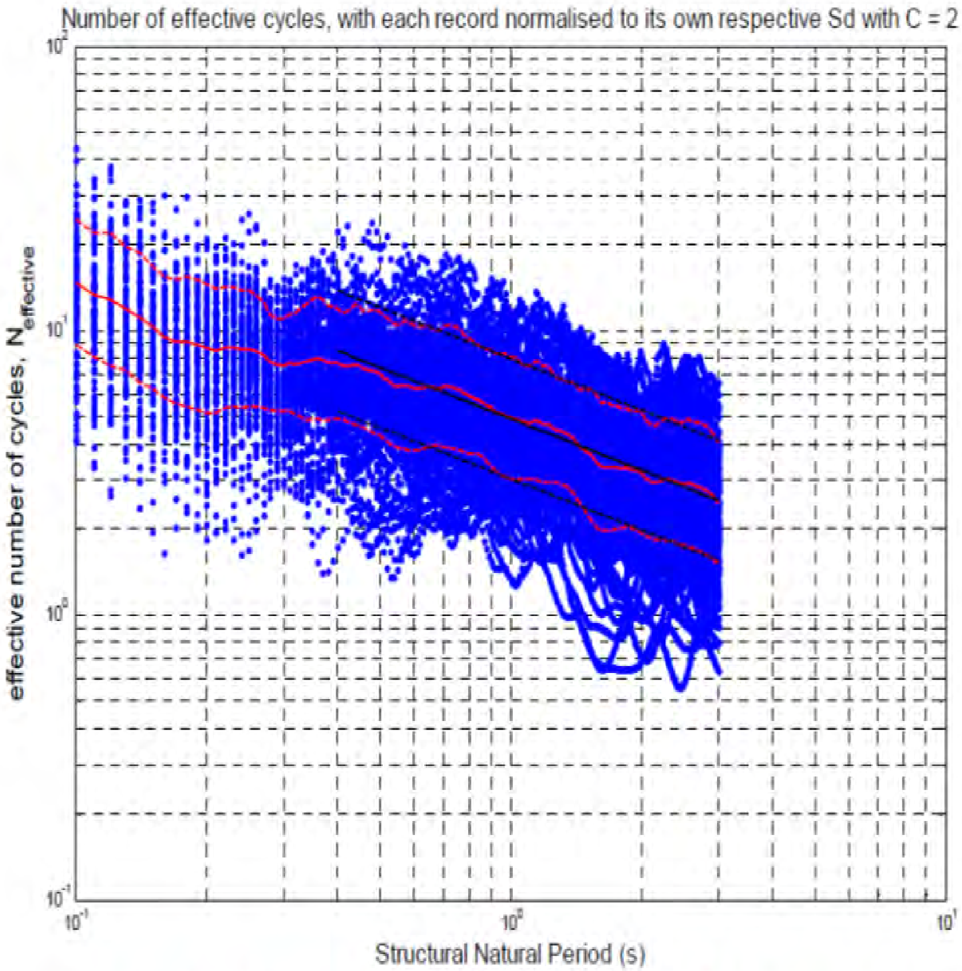


C = 1: For Concrete-critical fatigue



C = 2: Reinforcing-steel fatigue

Comparison:
Canterbury vs. World



C = 2: Reinforcing-steel fatigue

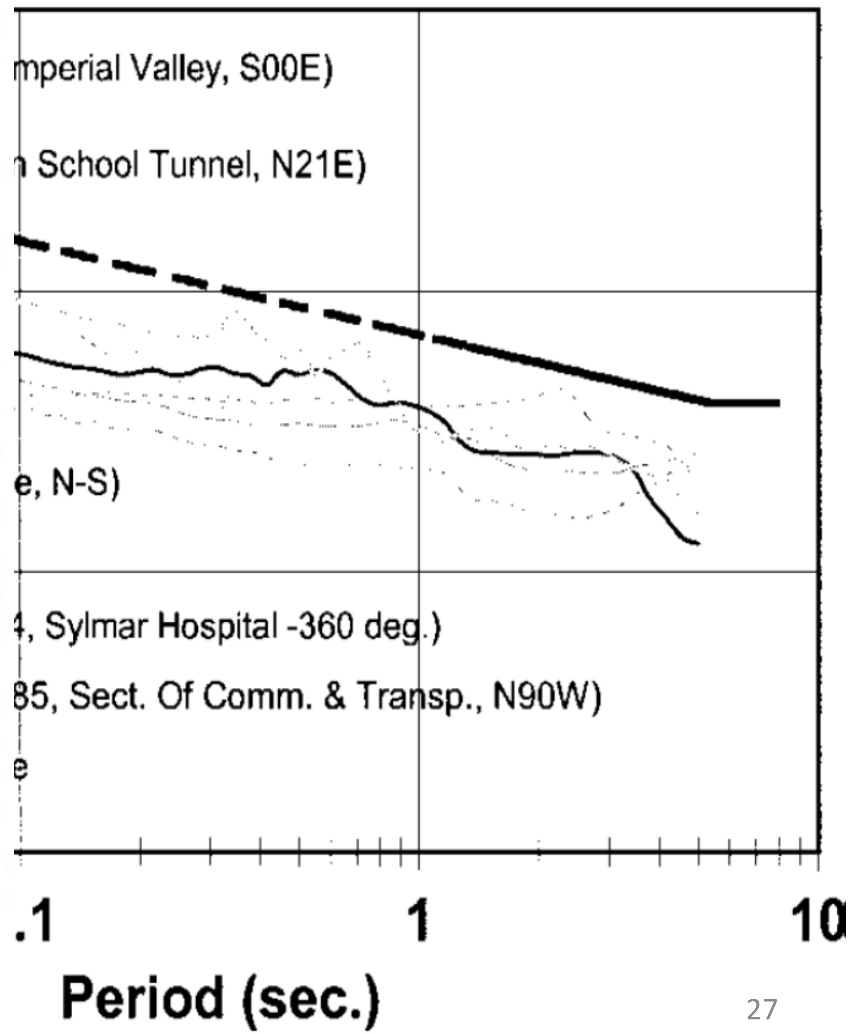
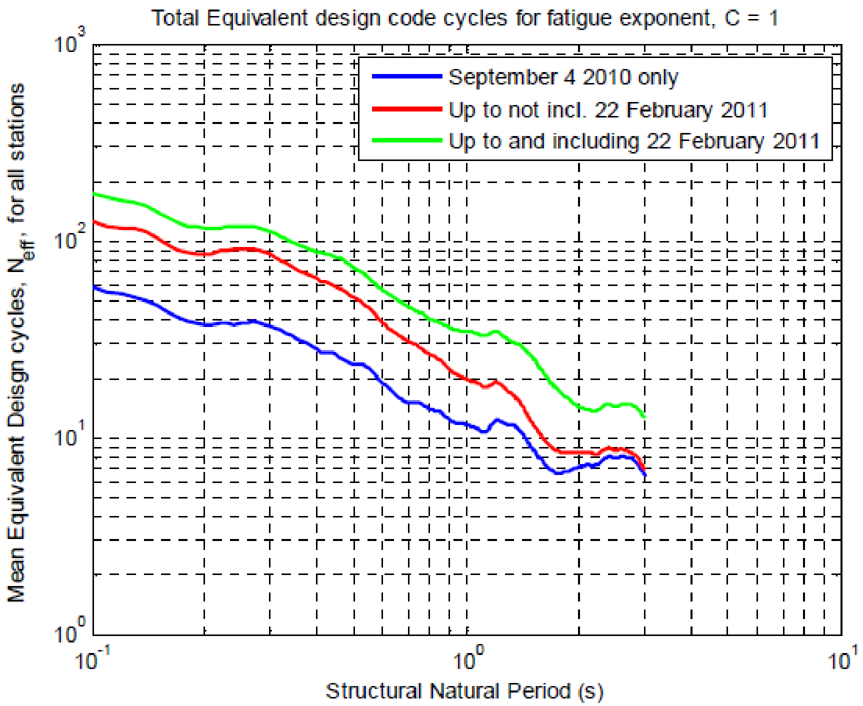
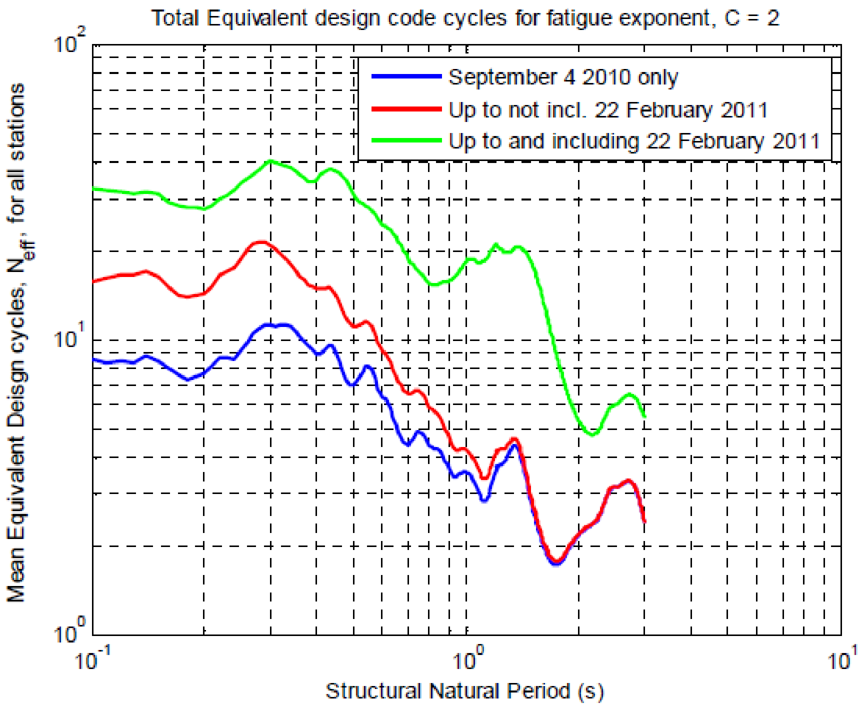


Figure 3: Cumulative Fatigue Demand Spectra: major (M4.9+) earthquakes from 4 September 2010 to 22 February 2011.



C = 1: For Concrete-critical fatigue



C = 2: Reinforcing-steel fatigue

Implications related to CTV Building

1. Older buildings could not be expected to survive *unscathed* when exposed to the multiple cyclic demands prior to and during the 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 classic ductile design in its current form is shown to be wanting.