TS 1170.5

Background to Spectral Shape Recommendations from SRWG

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University of Canterbury

Overview

A number of spectral shape options were considered by the SRWG.

An overview of the procedure followed and key results are described in the following slides.

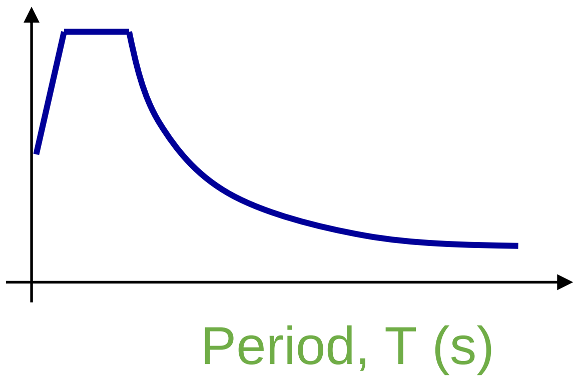
The final strength requirements are a result of the spectral shape, intensity and the design force expressions.

A later presentation will consider other factors, in addition to spectral shape, that were considered.

The likely impacts on the strength requirements will then be presented.

NZS1170.5 Horizontal Elastic Response Spectrum

$C(T) = C_h(T).Z.R.N(T,D)$



where, Z = hazard factor
R = return period factor.
N(T,D) = near-fault factor
$C_h(T)$ = spectral shape factor

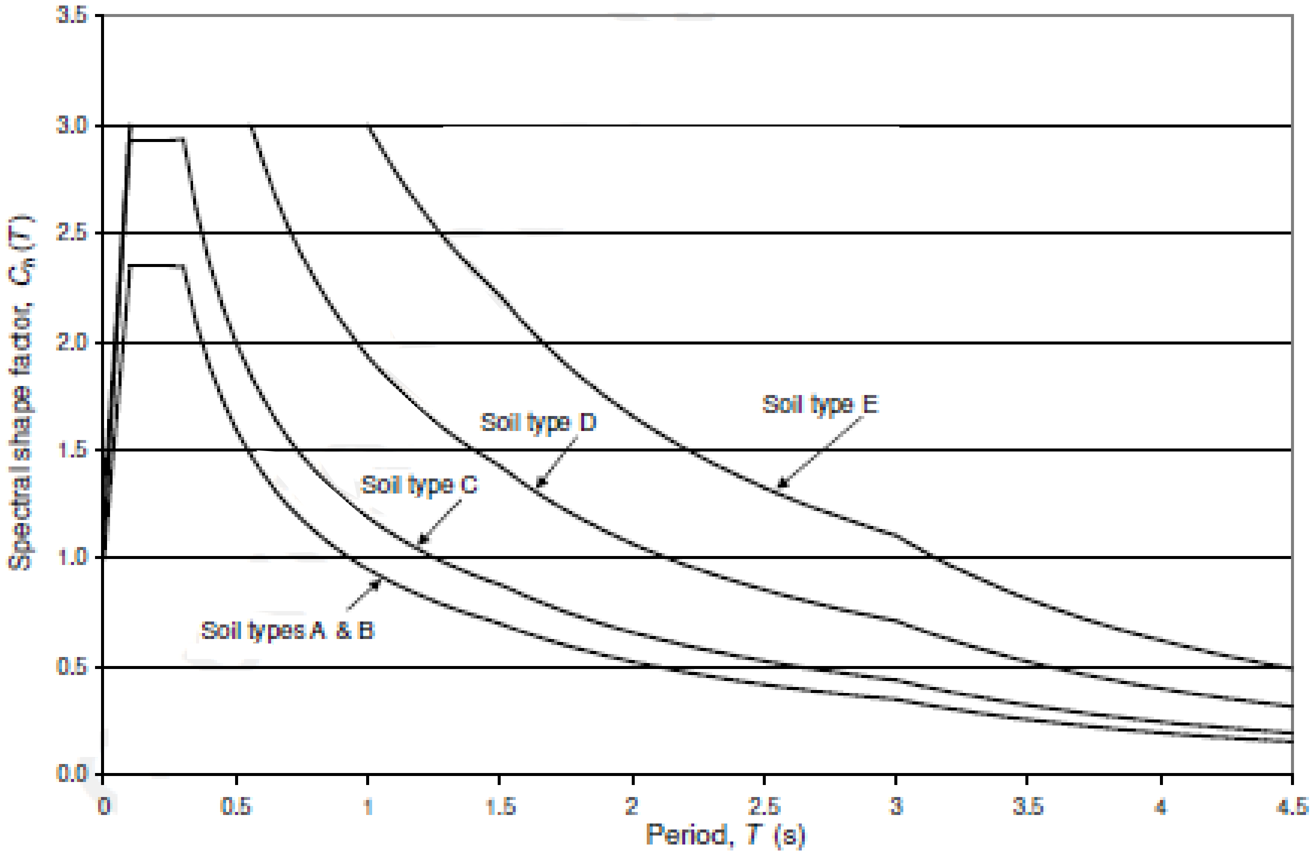


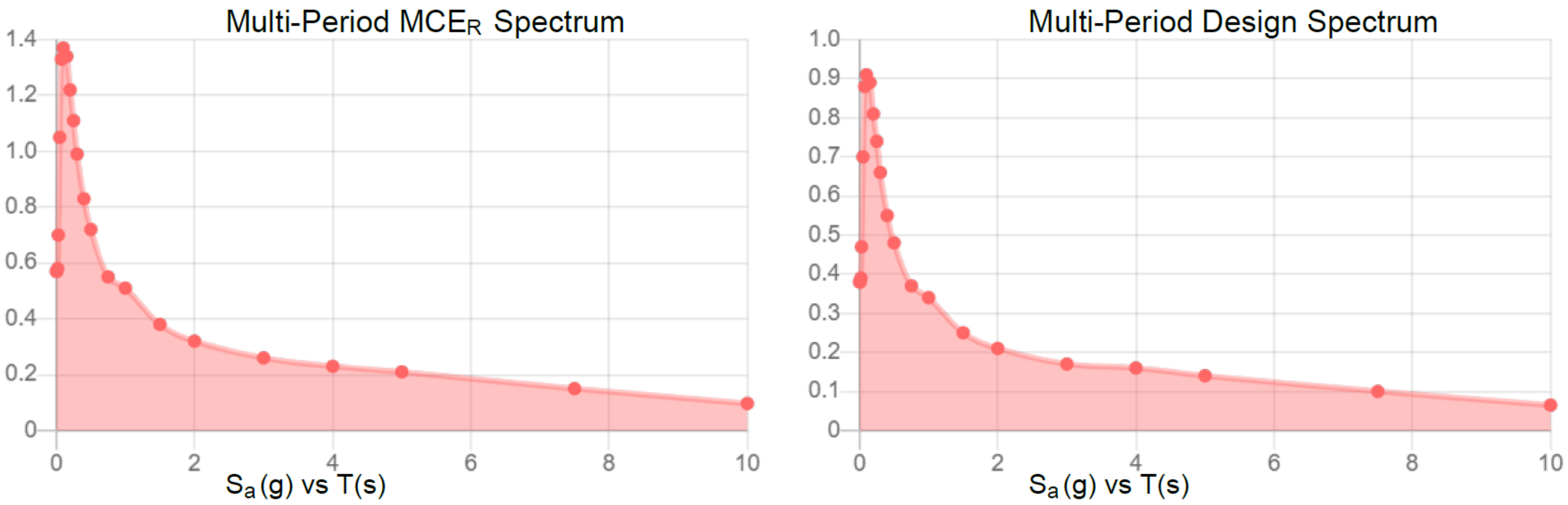
FIGURE 3.2 SPECTRAL SHAPE FACTOR, $C_h(T)$ FOR MODAL ANALYSIS, NUMERICAL INTEGRATION TIME HISTORY ANALYSIS, VERTICAL LOADING AND PARTS

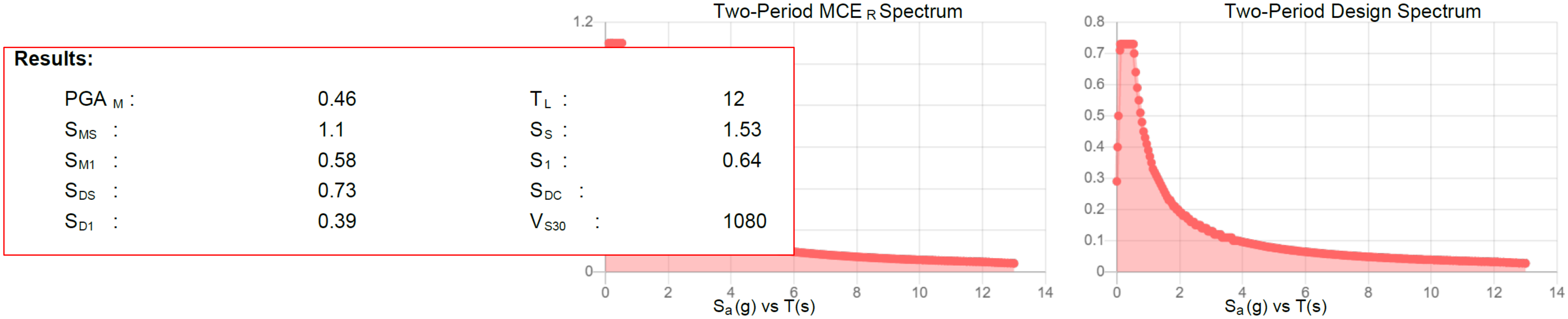
Spectral shape approaches used internationally?

US : ASCE7-22

Multi-point spectra.
See paper by Kircher & Rezaeian (2019)
Also:

https://asce7hazardtool.online/





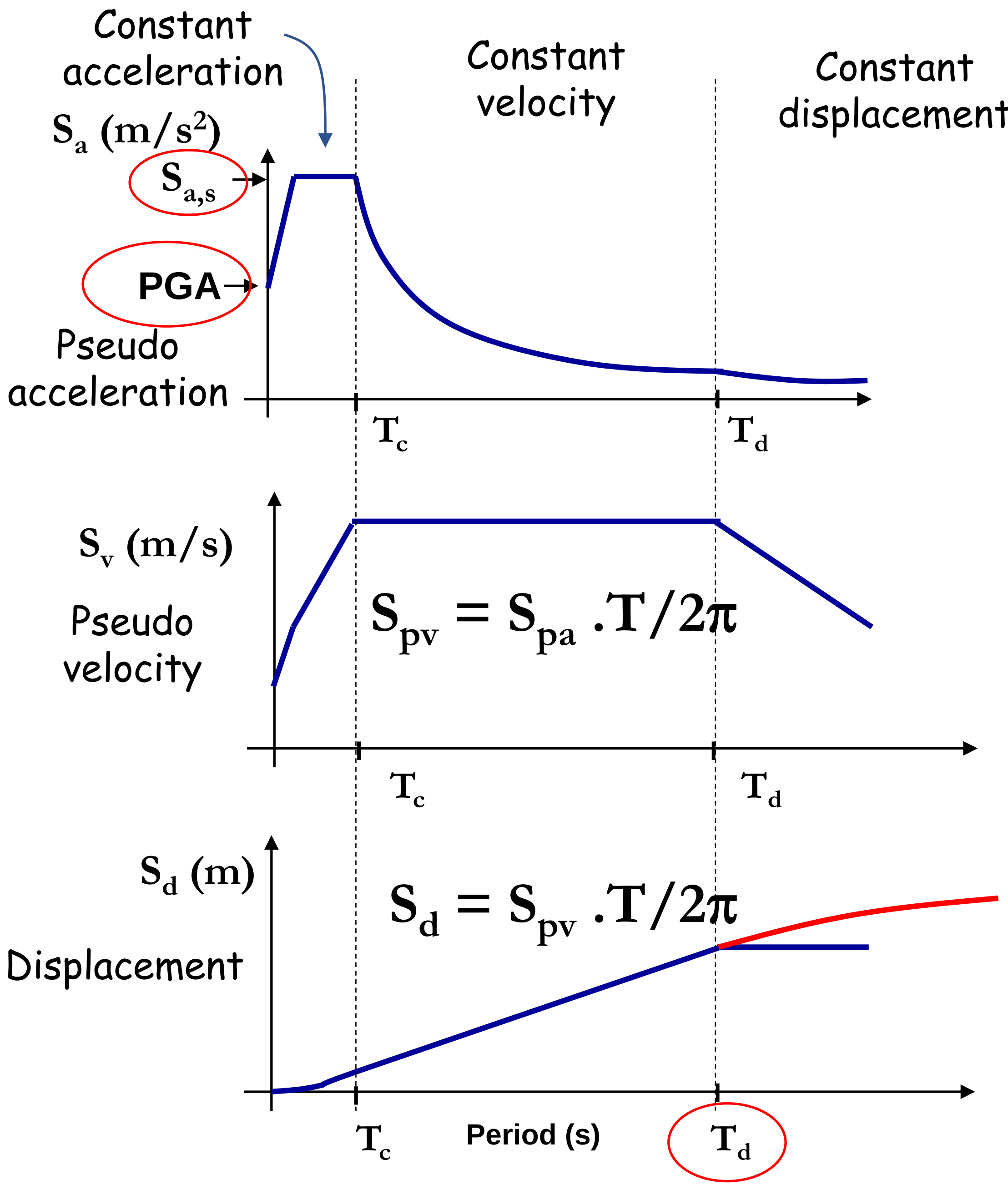
Results:

PGA_M :	0.46	T_L :	12
S_{MS} :	1.1	S_S :	1.53
S_{M1} :	0.58	S_1 :	0.64
S_{DS} :	0.73	S_{DC} :	
S_{D1} :	0.39	V_{S30} :	1080

Options for spectral shape considered by SRWG

Spectral Shape Method	Description
Method 0	Keep spectral shapes currently in NZS1170.5
Method 1	New spectral shapes that vary only according to site-soil class
Method 2	New spectral shapes that vary according to site-soil class and two intensity ranges (low and high intensity)
Method 3	New design spectral shapes that vary (in a continuous fashion) according to site-soil class and intensity
Method 4	Fit design spectral shape directly to location specific UHS or Nominal Risk Spectra
Method 5	Use (multipoint) UHS or Nominal Risk Spectra directly without fitting any spectral shape functions.

Options & equations for standardised design spectral shapes?



SRWG spectral shape equations:

$S_a(T) = PGA$ for T = 0s (7)

$S_a(T) = S_{a,s}$ for 0.1s < T < T_c (8)

$S_a(T) = S_{a,s}\frac{T_c}{T}$ for T_c < T < T_d (9)

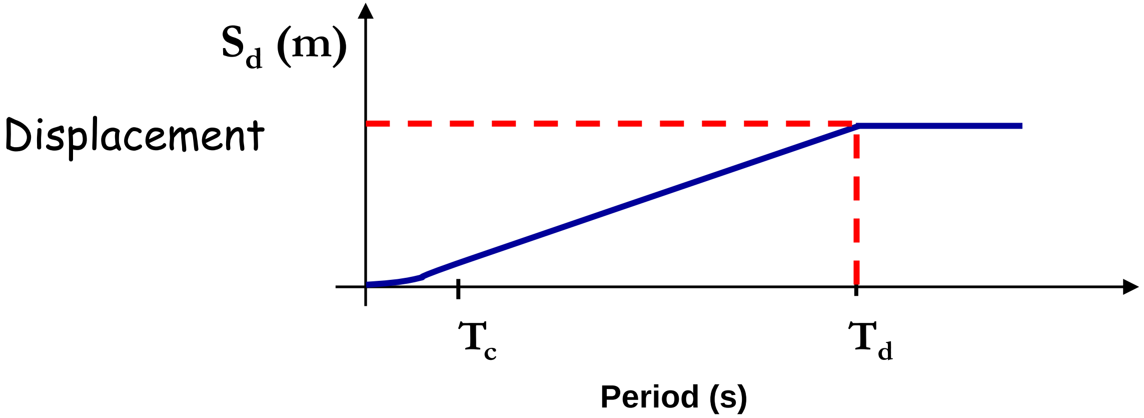
$S_a(T) = S_{a,s}\frac{T_c}{T}\left(\frac{T_d}{T}\right)^{0.5}$ for T_d < T (10)

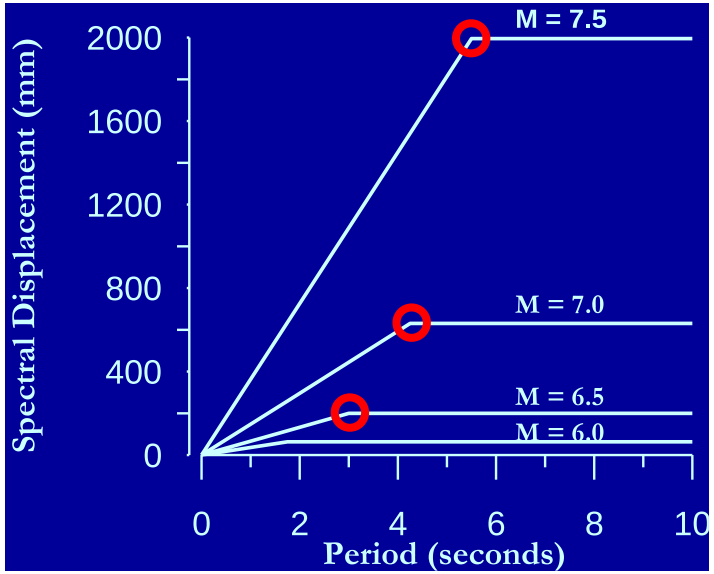
Why the new expression for S_a(T) at long periods, T > T_d ?

Spectral shape at long periods?

Faccioli et al. (2004) examined shape of spectra at long periods and noted:

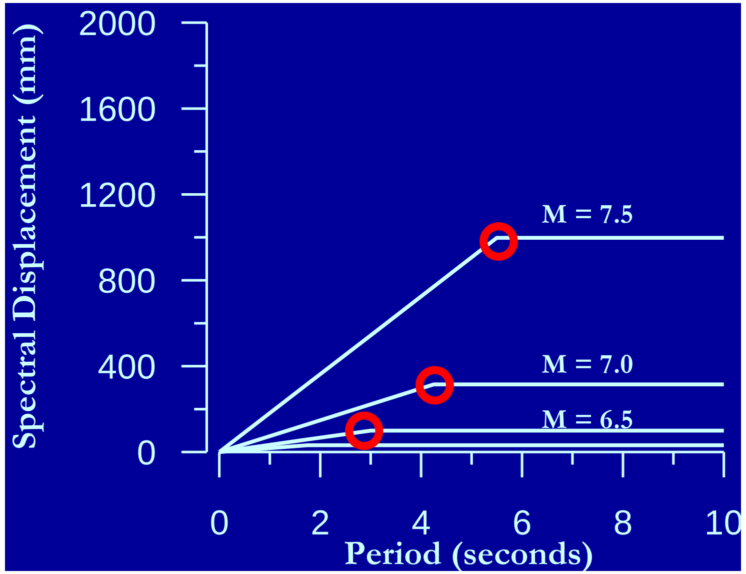
- At very long periods (e.g. T>10s), the spectral displacement demand corresponds to the peak ground displacement.
- The period, T_d, at which peak spectral displacement demand develops was found to be a function of earthquake magnitude, M.
- The peak value of the displacement depends on both the EQ magnitude and fault distance, r.





5% Damped Spectra at r = 10km

Figures from Priestley et al. (2007)



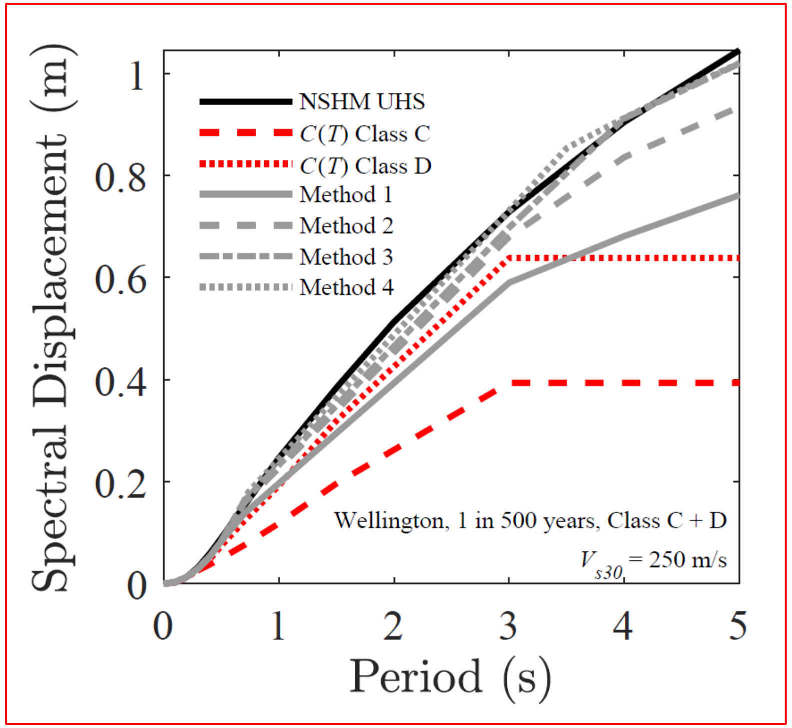
5% Damped spectra at r = 20 km

Relevance for shape of UHS at long periods?

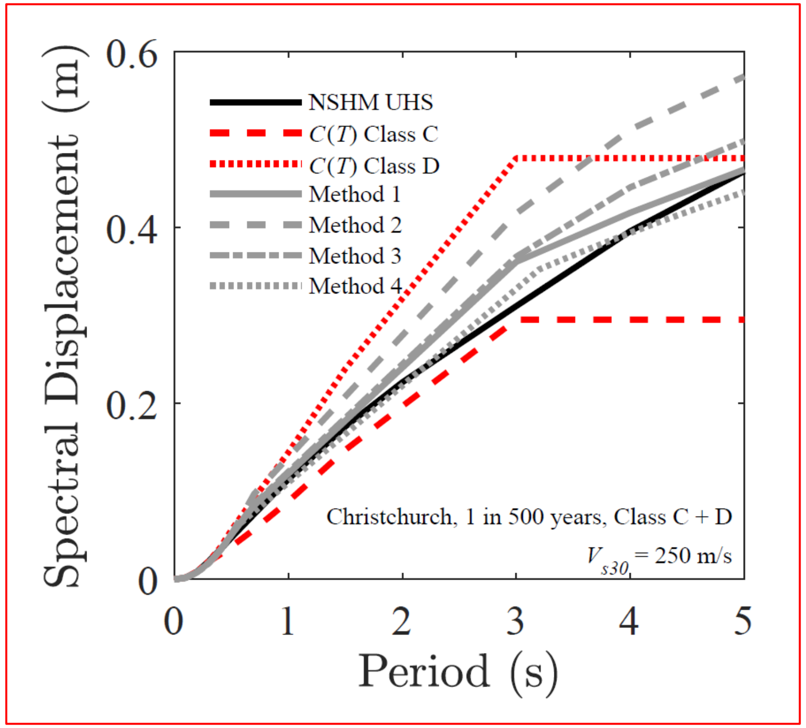
Uniform hazard spectra (UHS) demands are affected by a wide <u>range</u> of possible earthquake magnitudes, M, and distances, r.

→ hence, UHS don't typically exhibit a clear plateau in spectral displacement demands, even though demands at long periods do tend to flatten.

Wellington
T_R = 500yrs



Christchurch
T_R = 500yrs



The new equation for long period demands captures this flattening better than old.

Setting elastic design spectrum considering UHS?

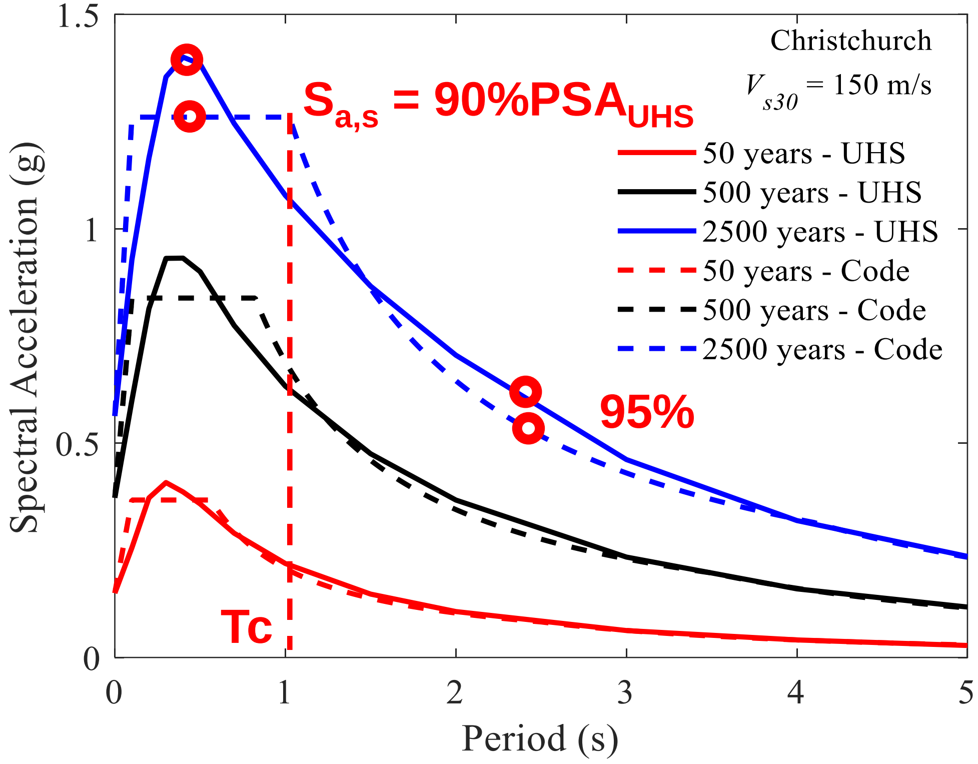
In addition to shape functions, need a procedure for fitting a design spectral shape to mean UHS (or notional risk spectrum).

SRWG procedure:

- The PGA is taken directly from the UHS data.
- The short period acceleration demand, $S_{a,s}$, is taken as 90% the peak spectral acceleration demand.
- The spectral acceleration corner period, T_c, is set by the following equation:

$$T_c = \frac{2\pi PSV}{S_{a,s}} \tag{11}$$

where PSV is the peak spectral velocity that has been taken equal to 95% the actual PSV from the UHS (in order to achieve a good fit with spectral demands across the medium period range).



- The spectral displacement corner period, T_d, was <u>initially</u> obtained via least-squares regression to minimise the difference between the UHS and design spectrum values for spectral velocity (between T_c and T_{max}) but then set to 3s as discussed later.

How much does the corner period, T_c, vary with intensity?

Values of T_c obtained for 12 cities, six Vs30 values and all return periods:

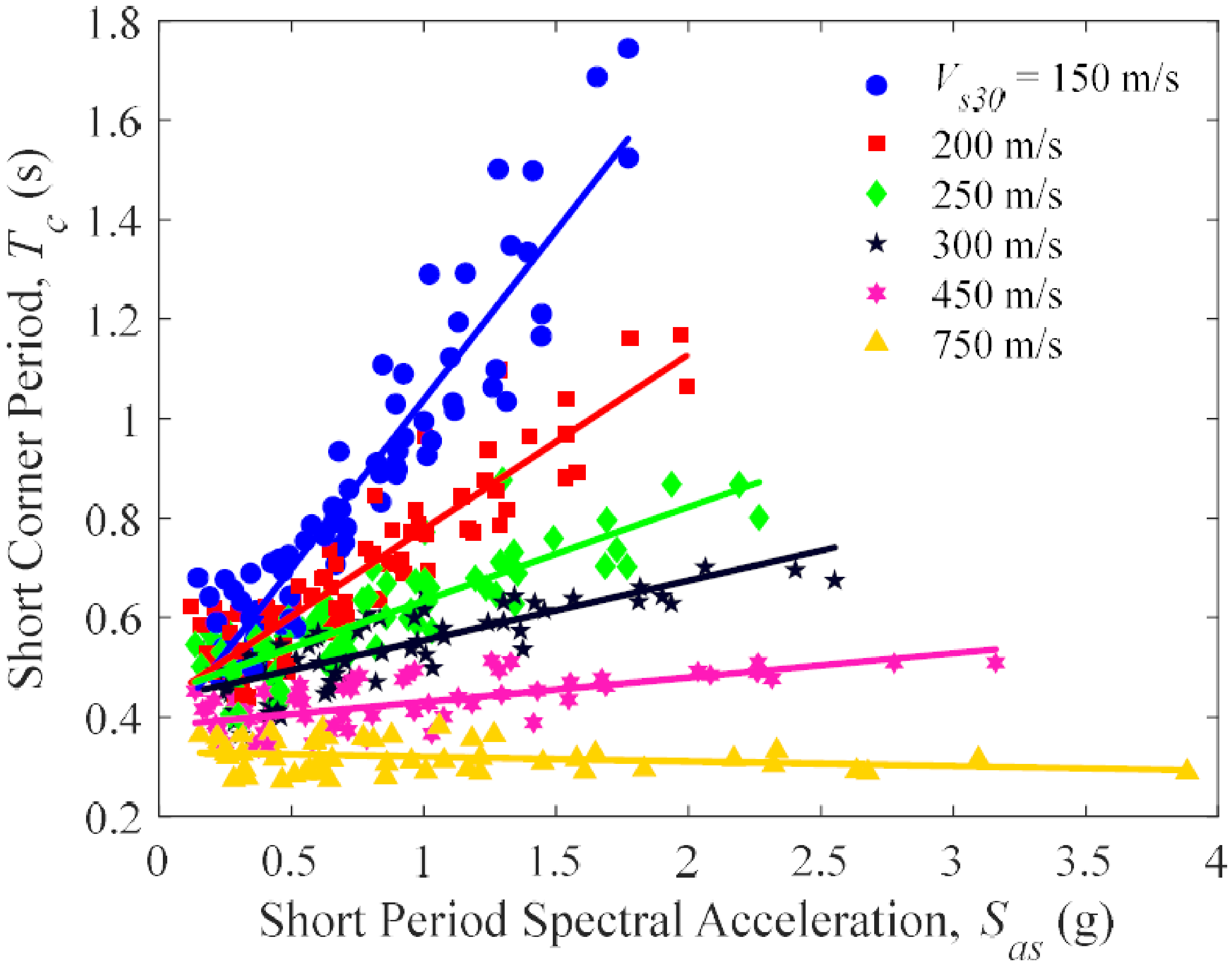
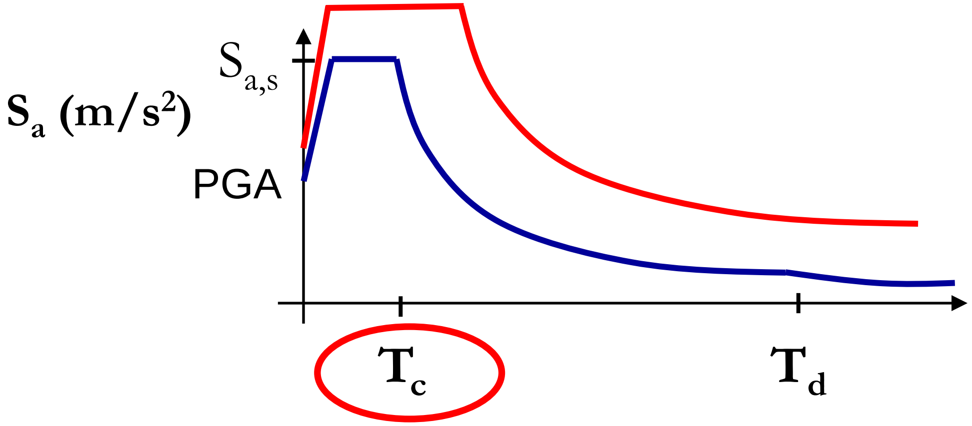


Figure 9: Variation of the short corner period as a function of short period spectral acceleration.



T_c seen to be a function of earthquake shaking intensity, as well as $V_{s,30}$ (site class)

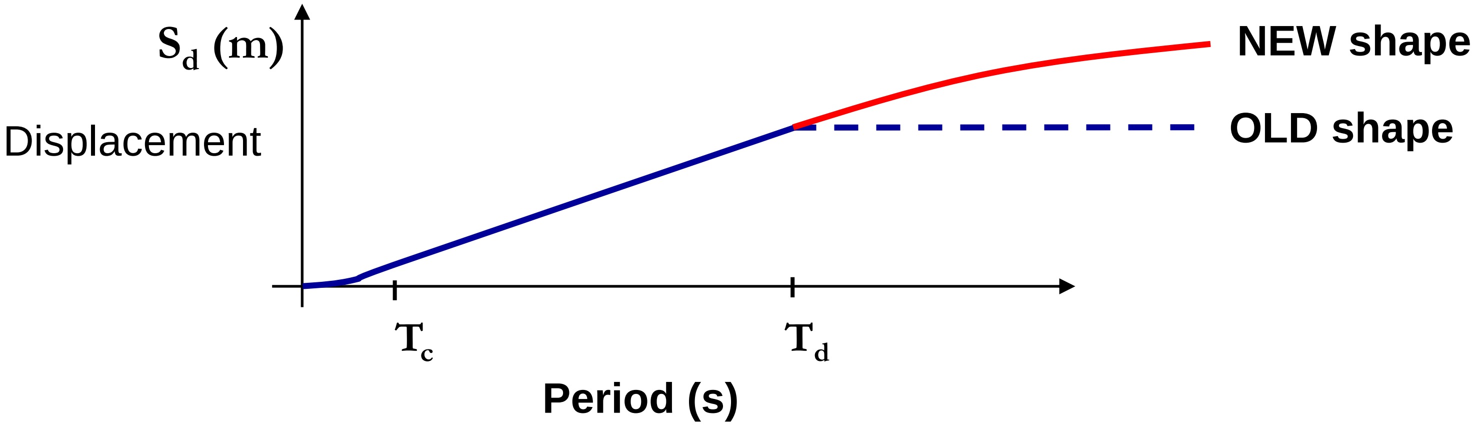
But what about the spectral velocity plateau corner period, T_d?

A desire was expressed to seek a single value of T_d if possible, for simplicity.

The new functional form for demands beyond T_d implies that accuracy of UHS fit is less sensitive to the value of T_d adopted.

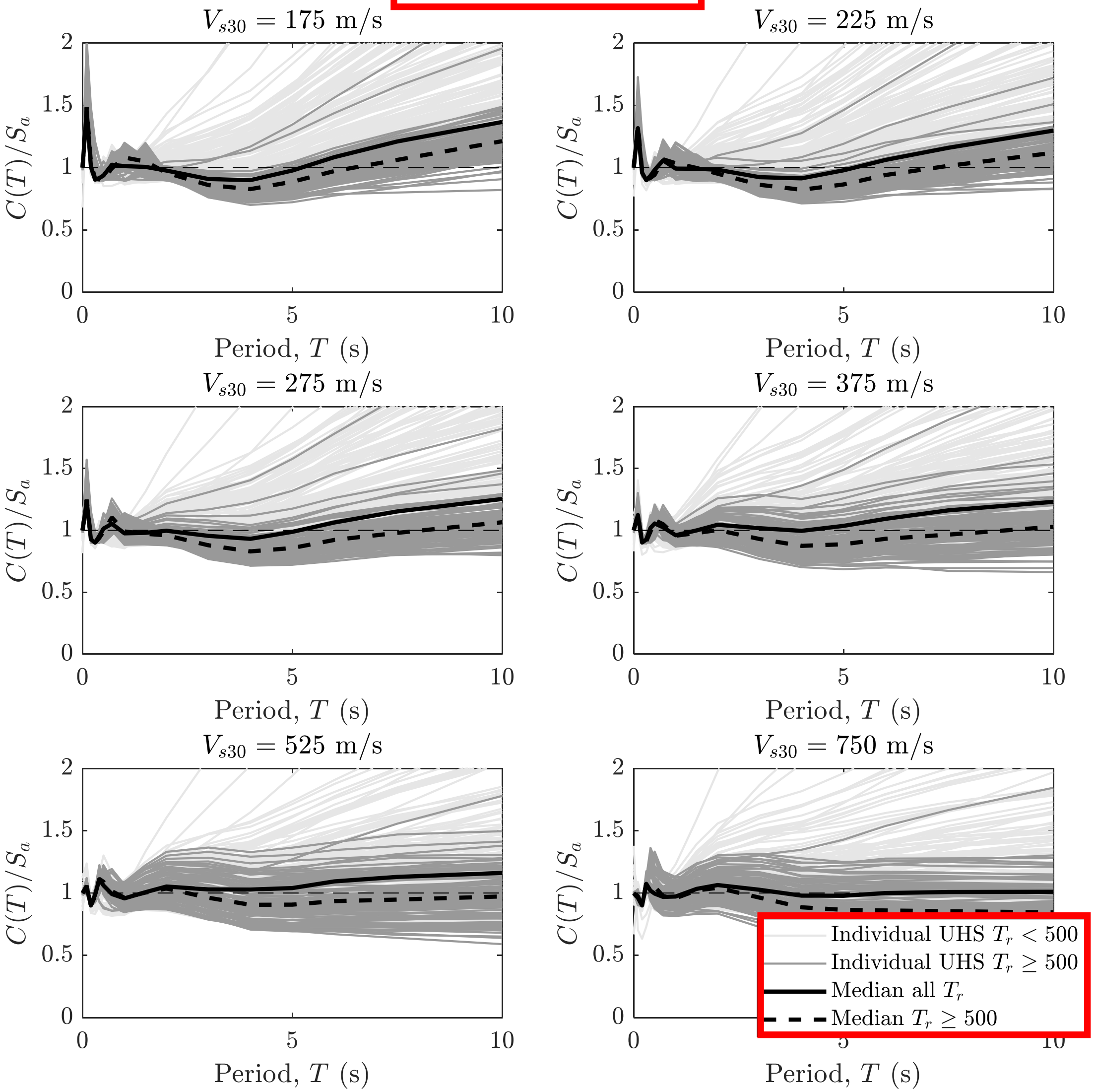
The following options for values of T_d values were trialled:

- $T_d = 2.0s$
- $T_d = 2.5s$
- $T_d = 3.0s$
- T_d = fitted site-by-site

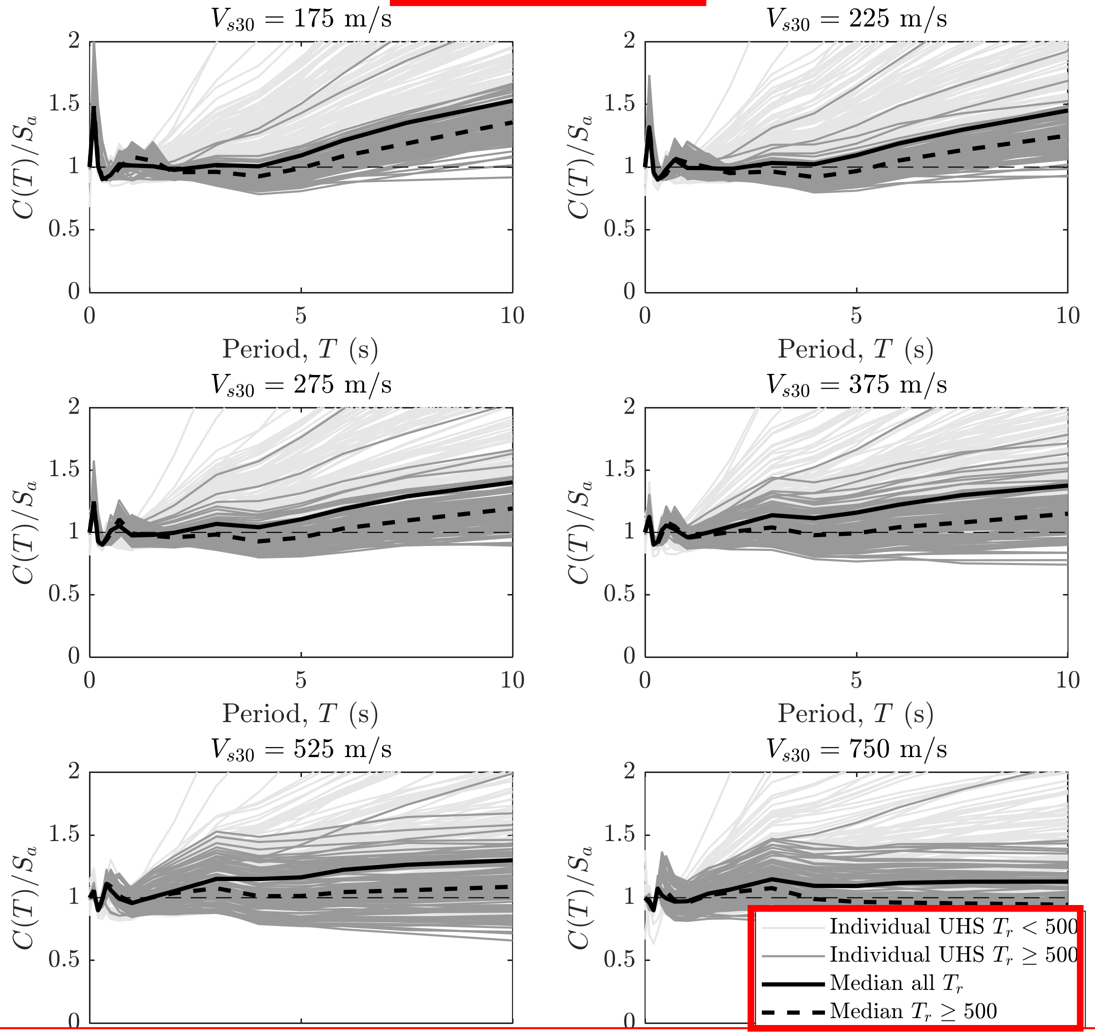


Ratio of design spectrum S_a to UHS value of S_a

$T_d = 2$ s



$T_d = 2.5$ s

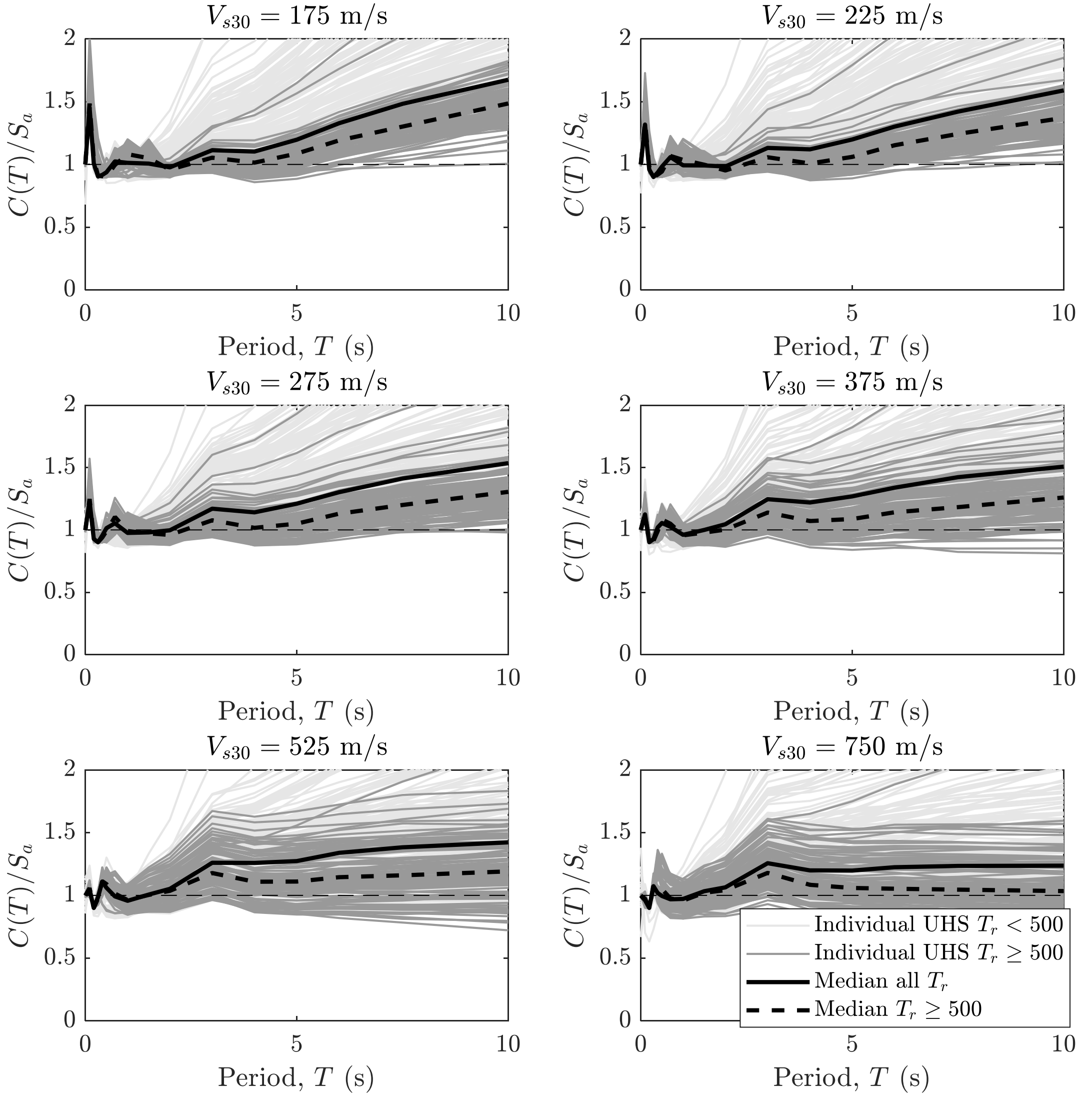


Things to note:

- Values > 1.0 indicate conservative design requirement

Ratio of design spectrum S_a to UHS value of S_a

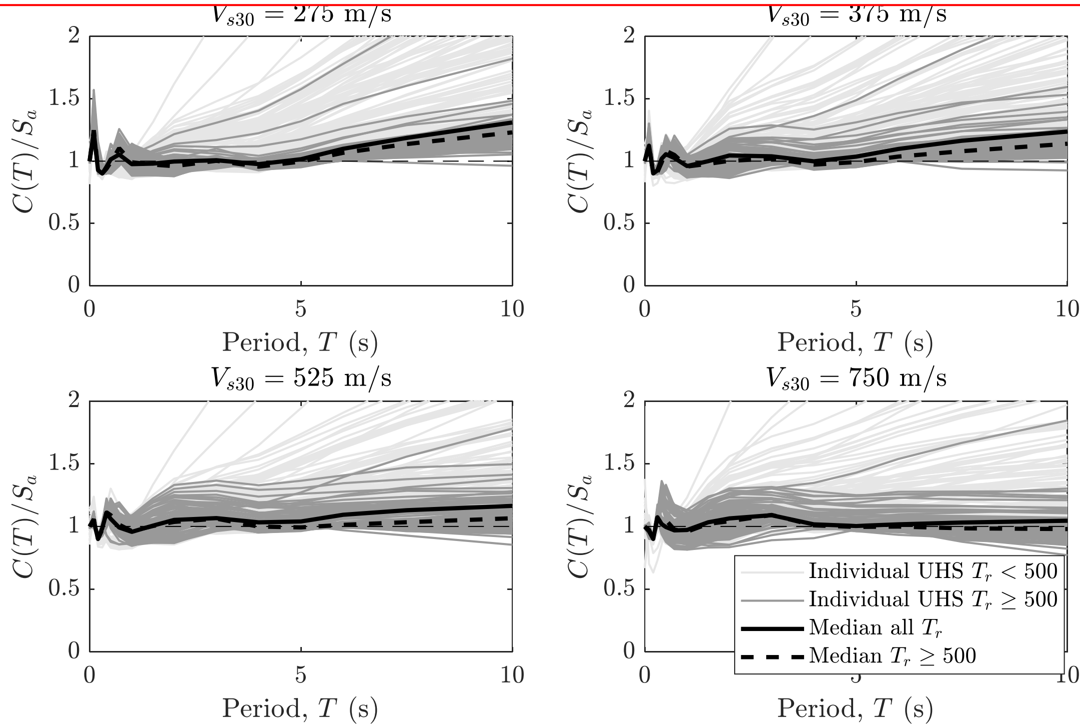
$T_d = 3$ s



T_d Fitted

The SRWG agreed to adopt T_d = 3s noting:

- Values > 1.0 indicate conservative design requirement
- Use of T_d = 3.0s is slightly conservative for most locations at ULS, out to T = 5.0 - 6.0s
- Use of T_d = 3.0s is clearly conservative for SLS (small return periods).



Evaluation of spectral shape options

Consider:

- How option could be implemented in a code
- Implications for strength requirements
- Implications for risk variability

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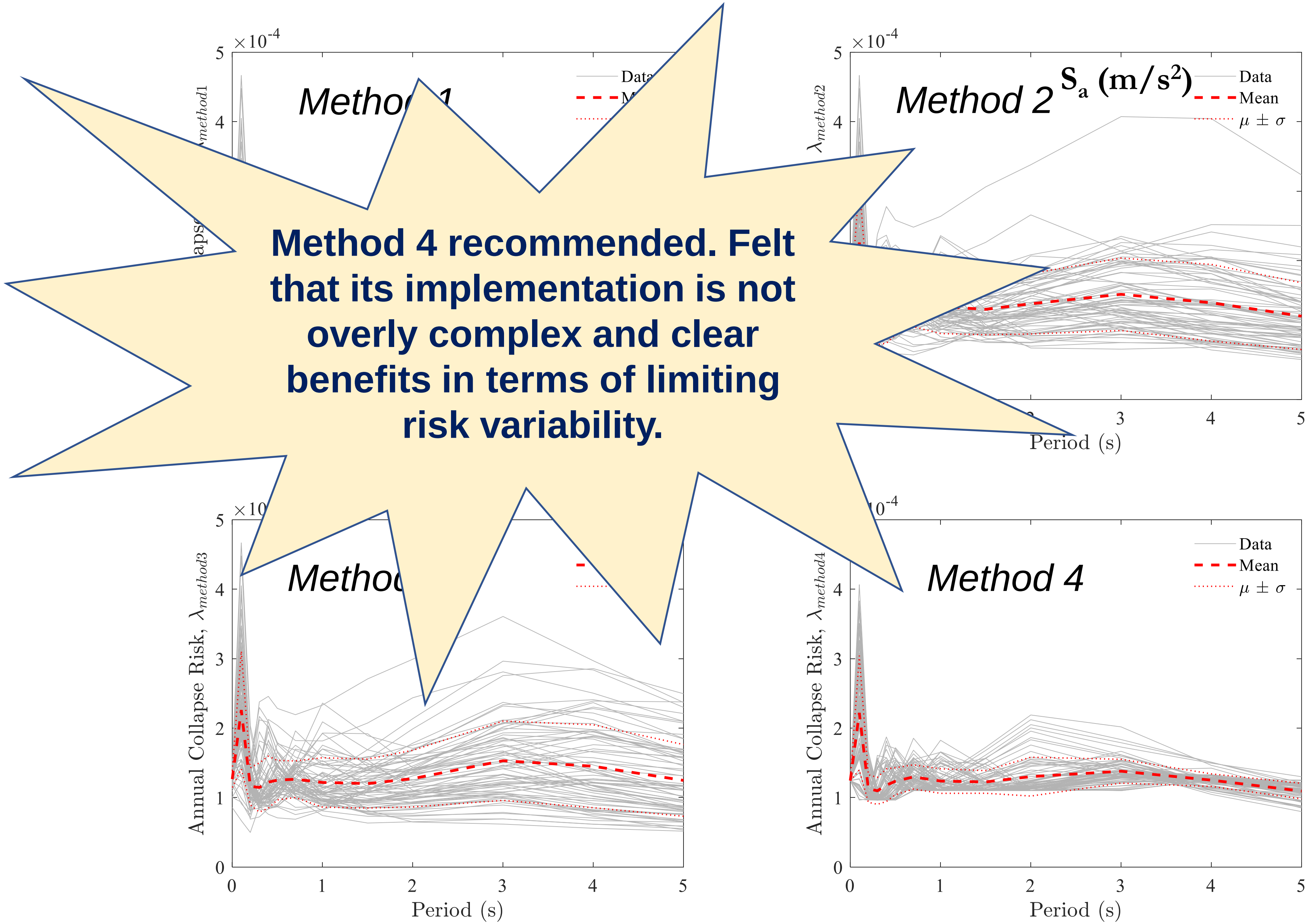
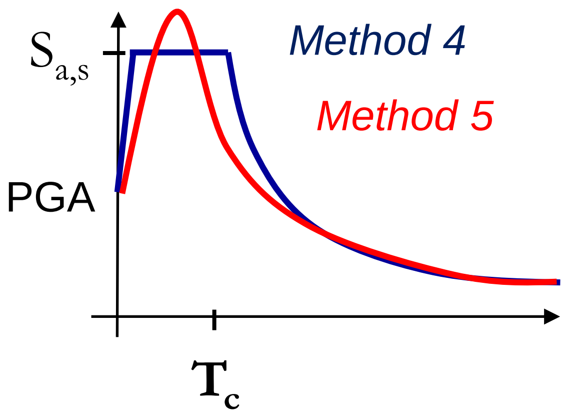
Approximate variability in risk due to spectral shape choices?



Method 4 recommended. Felt that its implementation is not overly complex and clear benefits in terms of limiting risk variability.



A benefit of Method 4 over Method 5 (smooth UHS) is that simplified spectral shape is less sensitive to period estimates in short period range.

Method 4 – using fitted UHS defined as a function of location, intensity (annual prob of exceedance) and site class

Implementation thought to be best via an on-line tool, similar to US/Italy etc. approaches.

Also identified that a fit spectrum could be defined according to each site class and return period with only three tabulated variables: PGA, S_{as} and T_c

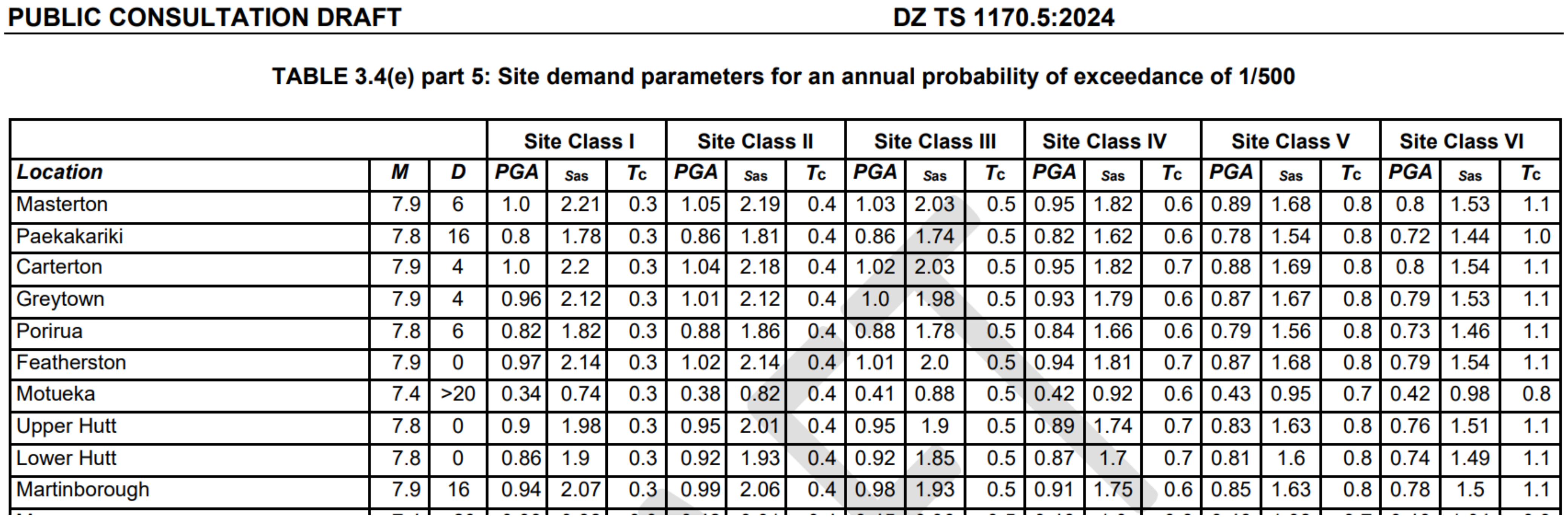


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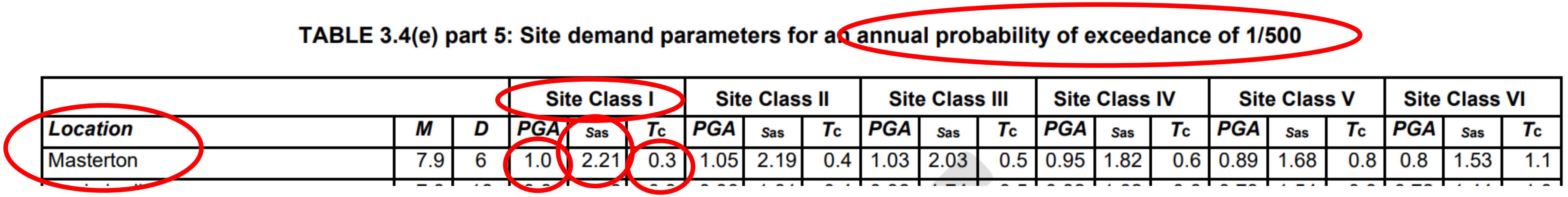
TABLE 3.4(e) part 5: Site demand parameters for an annual probability of exceedance of 1/500

			Site Class I			Site Class II			Site Class III			Site Class IV			Site Class V			Site Class VI		
Location	***M***	***D***	***PGA***	S_{as}	T_c	***PGA***	S_{as}	T_c	***PGA***	S_{as}	T_c	***PGA***	S_{as}	T_c	***PGA***	S_{as}	T_c	***PGA***	S_{as}	T_c
Masterton	7.9	6	1.0	2.21	0.3	1.05	2.19	0.4	1.03	2.03	0.5	0.95	1.82	0.6	0.89	1.68	0.8	0.8	1.53	1.1
Paekakariki	7.8	16	0.8	1.78	0.3	0.86	1.81	0.4	0.86	1.74	0.5	0.82	1.62	0.6	0.78	1.54	0.8	0.72	1.44	1.0
Carterton	7.9	4	1.0	2.2	0.3	1.04	2.18	0.4	1.02	2.03	0.5	0.95	1.82	0.7	0.88	1.69	0.8	0.8	1.54	1.1
Greytown	7.9	4	0.96	2.12	0.3	1.01	2.12	0.4	1.0	1.98	0.5	0.93	1.79	0.6	0.87	1.67	0.8	0.79	1.53	1.1
Porirua	7.8	6	0.82	1.82	0.3	0.88	1.86	0.4	0.88	1.78	0.5	0.84	1.66	0.6	0.79	1.56	0.8	0.73	1.46	1.1
Featherston	7.9	0	0.97	2.14	0.3	1.02	2.14	0.4	1.01	2.0	0.5	0.94	1.81	0.7	0.87	1.68	0.8	0.79	1.54	1.1
Motueka	7.4	>20	0.34	0.74	0.3	0.38	0.82	0.4	0.41	0.88	0.5	0.42	0.92	0.6	0.43	0.95	0.7	0.42	0.98	0.8
Upper Hutt	7.8	0	0.9	1.98	0.3	0.95	2.01	0.4	0.95	1.9	0.5	0.89	1.74	0.7	0.83	1.63	0.8	0.76	1.51	1.1
Lower Hutt	7.8	0	0.86	1.9	0.3	0.92	1.93	0.4	0.92	1.85	0.5	0.87	1.7	0.7	0.81	1.6	0.8	0.74	1.49	1.1
Martinborough	7.9	16	0.94	2.07	0.3	0.99	2.06	0.4	0.98	1.93	0.5	0.91	1.75	0.6	0.85	1.63	0.8	0.78	1.5	1.1

Table above are for a constant value of $T_d = 3.0s$

So how can we get spectral shape from TS1170.5?

Need: Location, Site class & Annual Probability of Exceedance.



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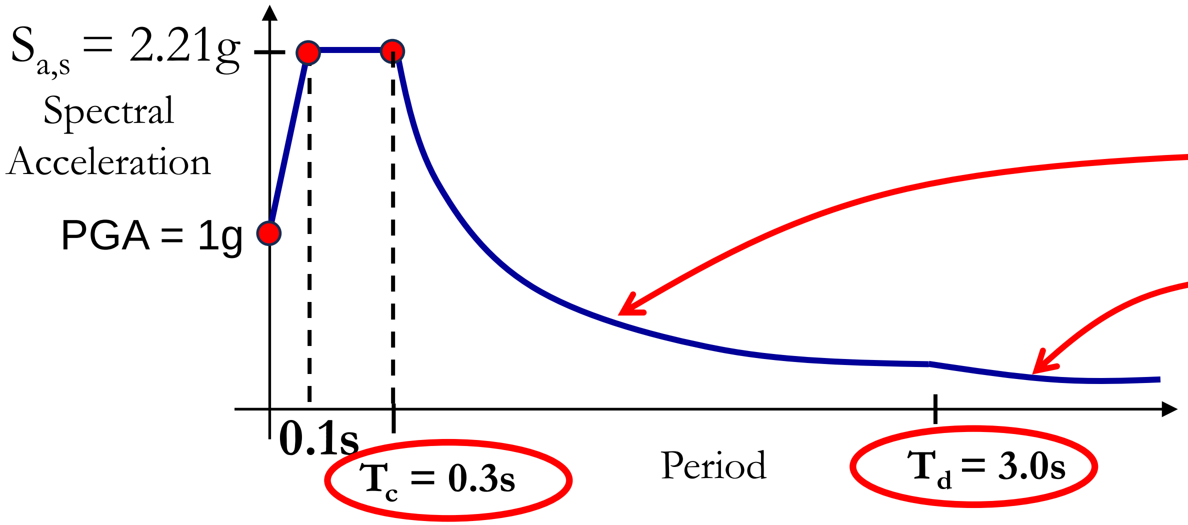


$S_a(T) = PGA$ for T = 0s

$S_a(T) = S_{a,s}$ for 0.1s < T < T_c

$S_a(T) = S_{a,s}\frac{T_c}{T}$ for T_c < T < T_d

$S_a(T) = S_{a,s}\frac{T_c}{T}\left(\frac{T_d}{T}\right)^{0.5}$ for T_d < T

Impact of new design spectra on design strength requirements?

Design strength expression

$$\boxed{V_{\mathrm{e}} = C_{\mathrm{d}}\left(T_1\right)W_{\mathrm{t}}}$$

$$\boxed{C_{\mathrm{d}}(T) = \frac{C(T)S_{\mathrm{p}}}{k_{\mu}}}$$

$$C(T) = S_a(T)N(T,D)$$

where

$S_a(T)$ = the spectral acceleration in units of g determined from Clause 3.1.2

$N(T,D)$ = the near-fault factor determined from Clause 3.1.4

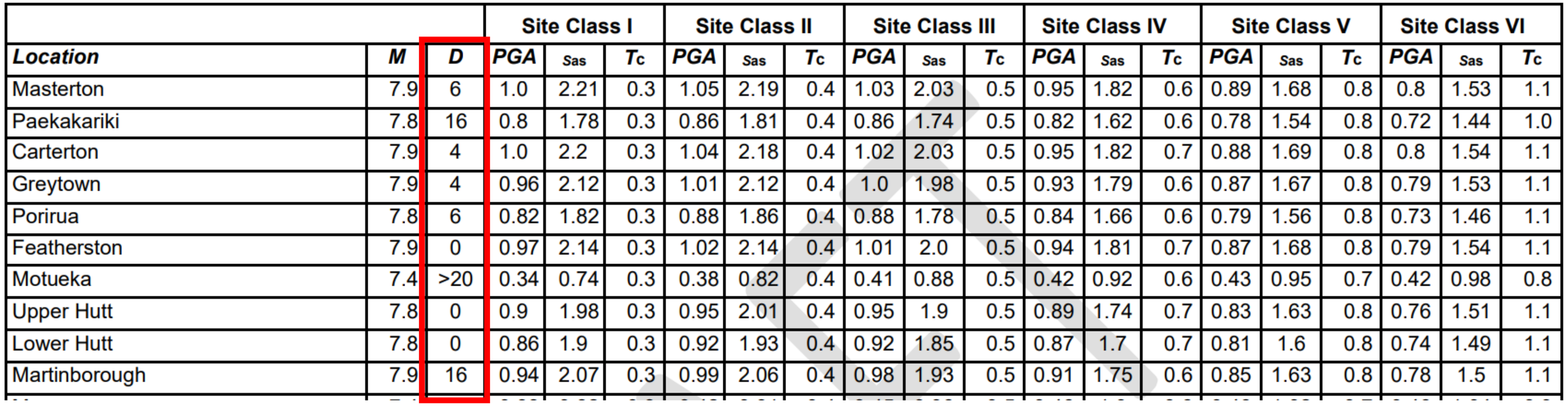
Near fault factor, N(T,D)?

The NSHM data does not explicitly allow for near-fault effects and directivity.

From a total risk perspective it is considered that the current approach for dealing with near fault effects is conservative (Weatherill, 2022). However, a suitable alternative has not been identified.

The SRWG recommends that near fault factors continue to be set as in NZS1170.5 (2004), except using updated distances D, and that a review of the provisions for near-fault and directivity effects be made in Phase II of the SRWG programme.

TABLE 3.4(e) part 5: Site demand parameters for an annual probability of exceedance of 1/500



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Upper Hutt	7.8	0	0.9	1.98	0.3	0.95	2.01	0.4	0.95	1.9	0.5	0.89	1.74	0.7	0.83	1.63	0.8	0.76	1.51	1.1
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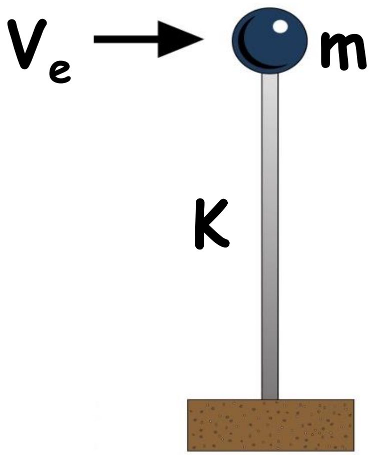
Martinborough	7.9	16	0.94	2.07	0.3	0.99	2.06	0.4	0.98	1.93	0.5	0.91	1.75	0.6	0.85	1.63	0.8	0.78	1.5	1.1

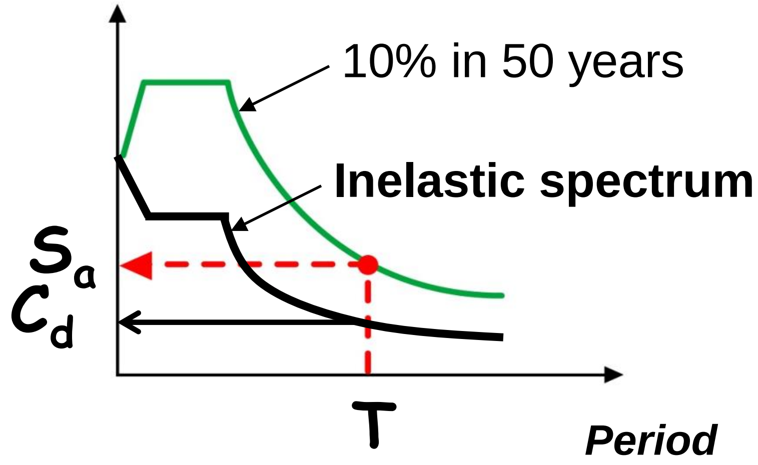
Allowing for inelastic deformation capacity via k_μ factor?

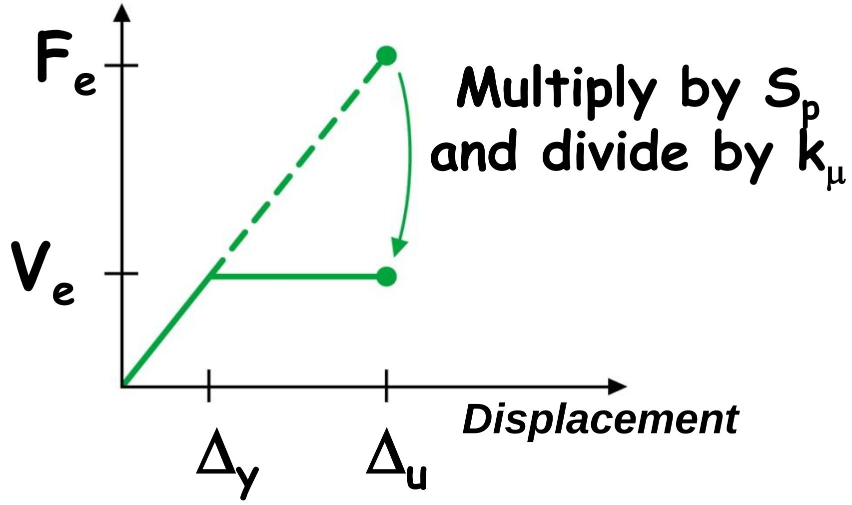
$V_e = W_t.C_d$

$V_e = W_t.C_d$



$$\boxed{C_d(T) = \frac{C(T)S_p}{k_\mu}}$$





Existing expressions for k_μ factor in NZS 1170.5 (2004)?

k_μ = inelastic spectrum scaling factor

for soil class A-D

$k_\mu = \mu$ for $T \geq 0.7$s

$= (\mu-1)T/0.7 + 1$ for $T < 0.7$s

for soil class E

$k_\mu = \mu$ for $T \geq 1$s or $\mu < 1.5$

$= (\mu-1.5)T + 1.5$ for $T < 1$s and $\mu \geq 1.5$

Note: T shall not be taken less than 0.4s for the purpose of calculating k_μ

Could a larger k_μ factor be justified for short periods?

The SRWG generally felt it was not feasible to re-examine k_μ factors and structural analysis provisions in this first set of revisions.

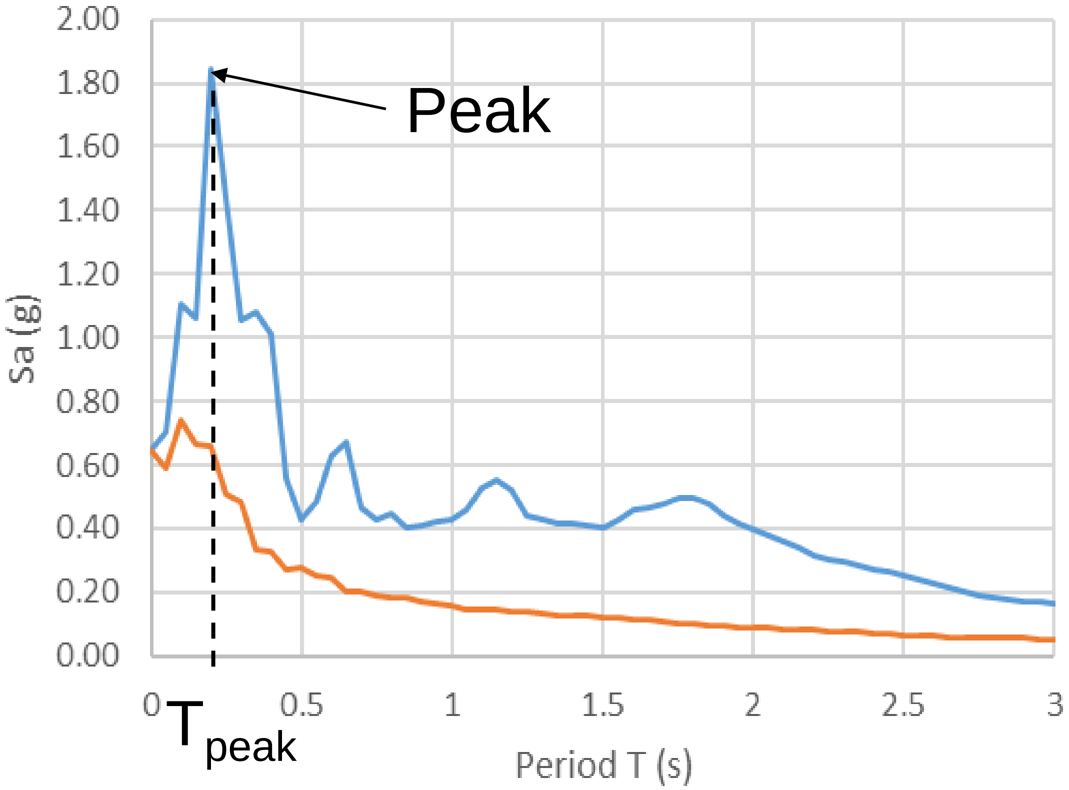
However, the SRWG were also concerned that demands on short-period buildings resulting from the new NSHM could be unreasonably high.

Could a larger k_μ factor be justified at short periods?

Beyond what period of vibration could the equal-displacement rule be applied?

It is currently applied for $T > 0.7s$ for most site classes in NZS 1170.5 (2004)

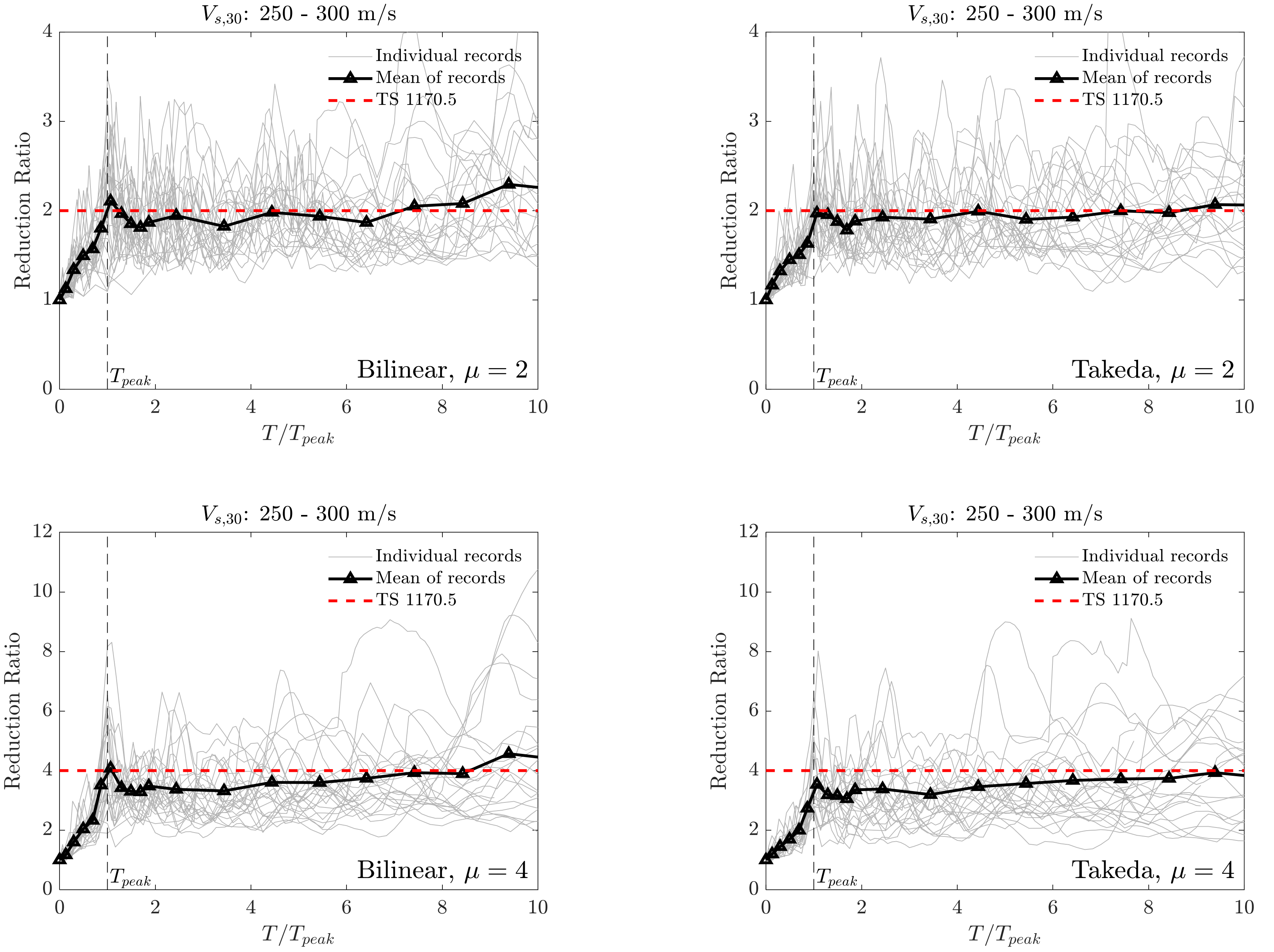
→ Consider apparent k_μ factors from inelastic spectra plotting T relative to a peak period, T_{peak}.



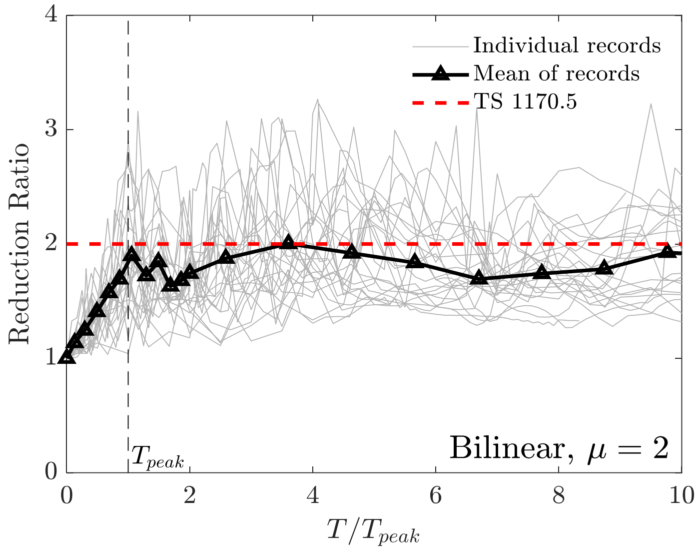
Investigation of demands using suite of records

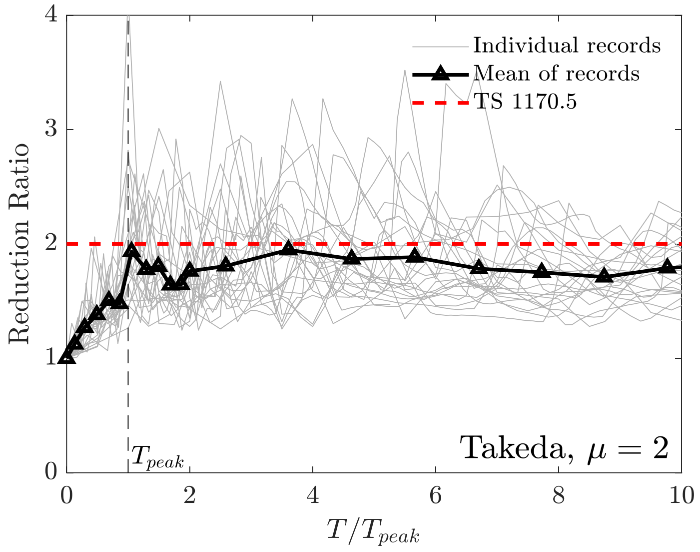
- Suite of records from PEER NGA-West2 Database
- $M_w > 5.5$
- Pulse-like records excluded
- Inelastic spectra developed using INSPECT (Carr) with following assumptions:
  - Bi-linear and Takeda hysteretic models with post-yield stiffness ratio, r = 0.05
  - Takeda parameters: Emori Schonbrich reloading with alpha = 0.5 and beta = 0.0
  - Secant proportional damping ratio 5%.

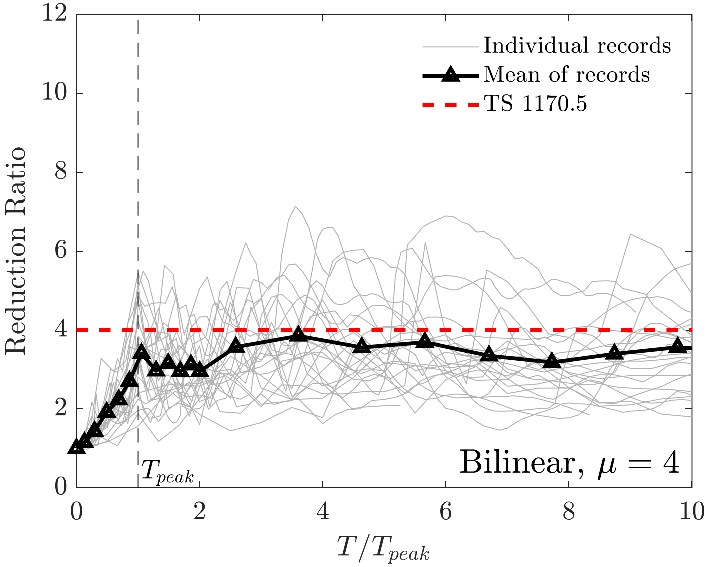
Soft soil sites: V_{s30} = 250 m/s – 300 m/s

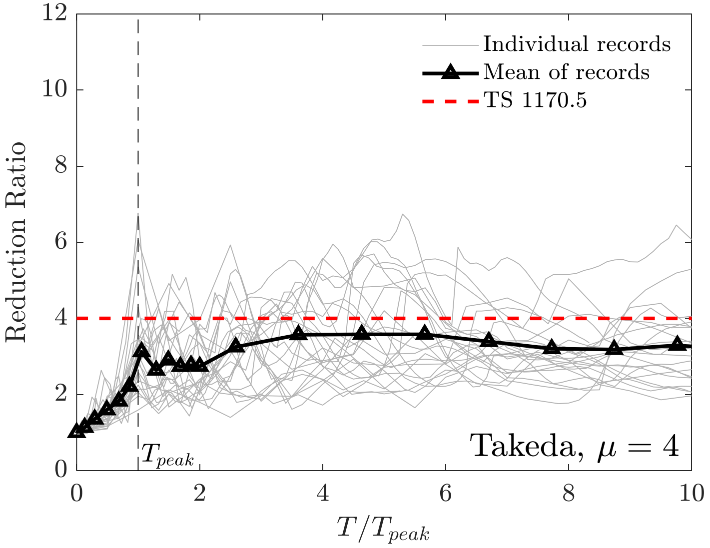


Rock sites: V_{s30} = 700 m/s – 2000 m/s







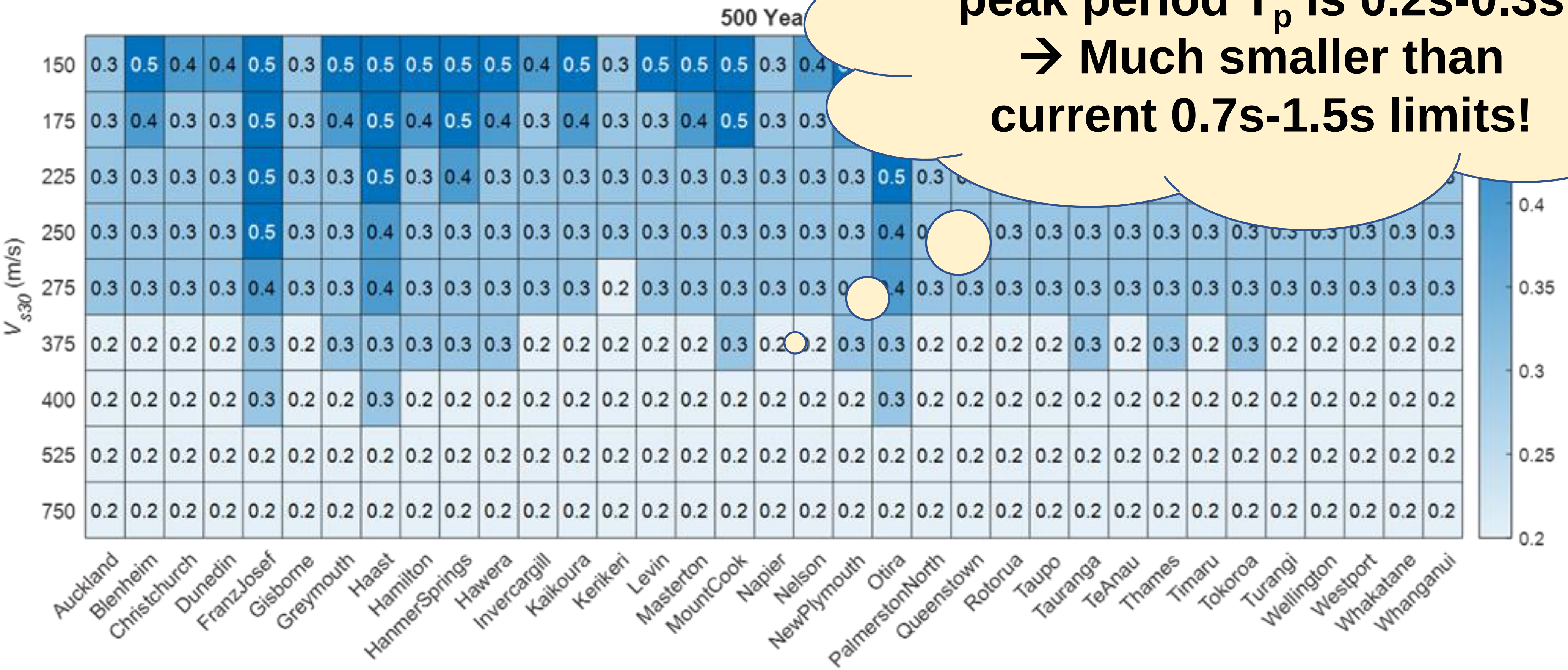


New period range for use of equal-displacement rule

Findings indicate that the equal displacement rule ($k_\mu = \mu$) is reasonable for periods greater than a peak period, T_p (a fraction of T_c).

What is value of T_p for different sites?

For most locations, the peak period T_p is 0.2s-0.3s
→ Much smaller than current 0.7s-1.5s limits!



V_{s30} (m/s)	Auckland	Blenheim	Christchurch	Dunedin	FranzJosef	Gisborne	Greymouth	Haast	Hamilton	HanmerSprings	Hawera	Invercargill	Kaikoura	Kerikeri	Levin	Masterton	MountCook	Napier	Nelson	NewPlymouth	Otira	PalmerstonNorth	Queenstown	Rotorua	Taupo	Tauranga	TeAnau	Thames	Timaru	Tokoroa	Turangi	Wellington	Westport	Whakatane	Whanganui
150	0.3	0.5	0.4	0.4	0.5	0.3	0.5	0.5	0.5	0.5	0.5	0.4	0.5	0.3	0.5	0.5	0.5	0.3	0.4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
175	0.3	0.4	0.3	0.3	0.5	0.3	0.4	0.5	0.4	0.5	0.4	0.3	0.4	0.3	0.3	0.4	0.5	0.3	0.3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
225	0.3	0.3	0.3	0.3	0.5	0.3	0.3	0.5	0.3	0.4	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.5	0.3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
250	0.3	0.3	0.3	0.3	0.5	0.3	0.3	0.4	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.4	[illegible]	[illegible]	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
275	0.3	0.3	0.3	0.3	0.4	0.3	0.3	0.4	0.3	0.3	0.3	0.3	0.3	0.2	0.3	0.3	0.3	0.3	0.3	[illegible]	0.4	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
375	0.2	0.2	0.2	0.2	0.3	0.2	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.2	0.3	0.2	0.2	0.3	0.3	0.2	0.2	0.2	0.2	0.3	0.2	0.3	0.2	0.3	0.2	0.2	0.2	0.2	0.2
400	0.2	0.2	0.2	0.2	0.3	0.2	0.2	0.3	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
525	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
750	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2

New expressions for k_μ factor?

Initial suggestion:

~~$k_\mu = \mu$ for $T \geq T_c/2$~~

~~$= (\mu-1)T/0.7 + 1$ for $T < T_c/2$~~

$$\boxed{k_\mu = \mu}$$

Because the value of T_{peak} is low and because members of the SRWG feel that there is some anecdotal evidence that short period buildings are already very resilient, the SRWG is recommending $k_\mu = \mu$ for all periods of vibration.

A recent study in US (FEMA P-2139-1, 2020) also found that short-period buildings are not as prone to collapse as traditional design provisions might suggest.

Key reasons:

- Design ductility values and associated displacement capacities for timber-framed buildings, steel CBF buildings and reinforced masonry buildings are too conservative for short period buildings.
- A small lateral deformation of foundations (not accounted for in structural analyses) could significantly increase the period & total deformation capacity.

Design strength implications?

$$V_e = C_d(T_1)W_t$$

The following slides present the resulting design coefficients, $C_d(T)$, for some of the main cities around New Zealand.

$$\boxed{C_d(T) = \frac{C(T)S_p}{k_\mu}}$$

Results are for Tr = 500 years (ULS).

Results show elastic demands and demands for $\mu = 4.0$.

Design coefficients are compared with the existing NZS 1170.5 (2004) provisions (but with uncertainty in equivalence of site classes)

Soil Classification Compatibility for Inelastic Spectra Comparison

1. NZS1170.5 site classes encompass a wide range of $V_s(30)$ values
   (e.g., Site Class C in WEL has Vs(30)=200-500m/s)
2. For a given Vs(30) value, several different site classes could be specified in NZS1170.5
   (e.g., Vs(30)=250m/s can be Class C, D or even E)
3. The range of Vs(30) values corresponding to a given site class in NZS1170.5 is region-(location)-dependent
   (e.g., typically for Site Class D: Vs(30)=175-225 m/s in CHC; but is Vs(30)=200-350m/s in WEL).

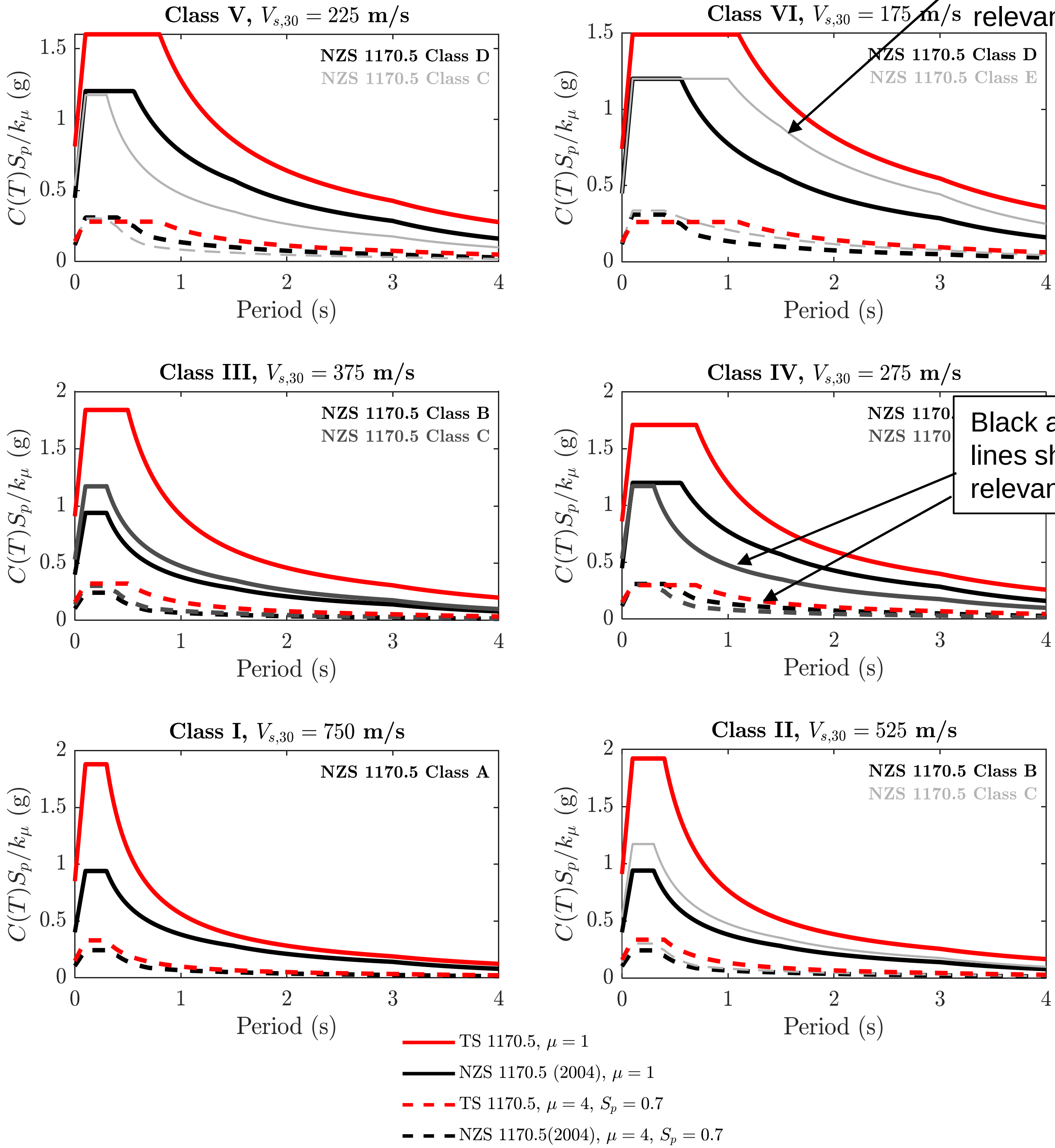
Given the above (courtesy of Misko), inelastic spectra comparisons are made considering the soil classification relationships shown in the table below:

(a) Site classes in black are equally appropriate

(b) Site classes in red are less likely in relative terms but still relevant

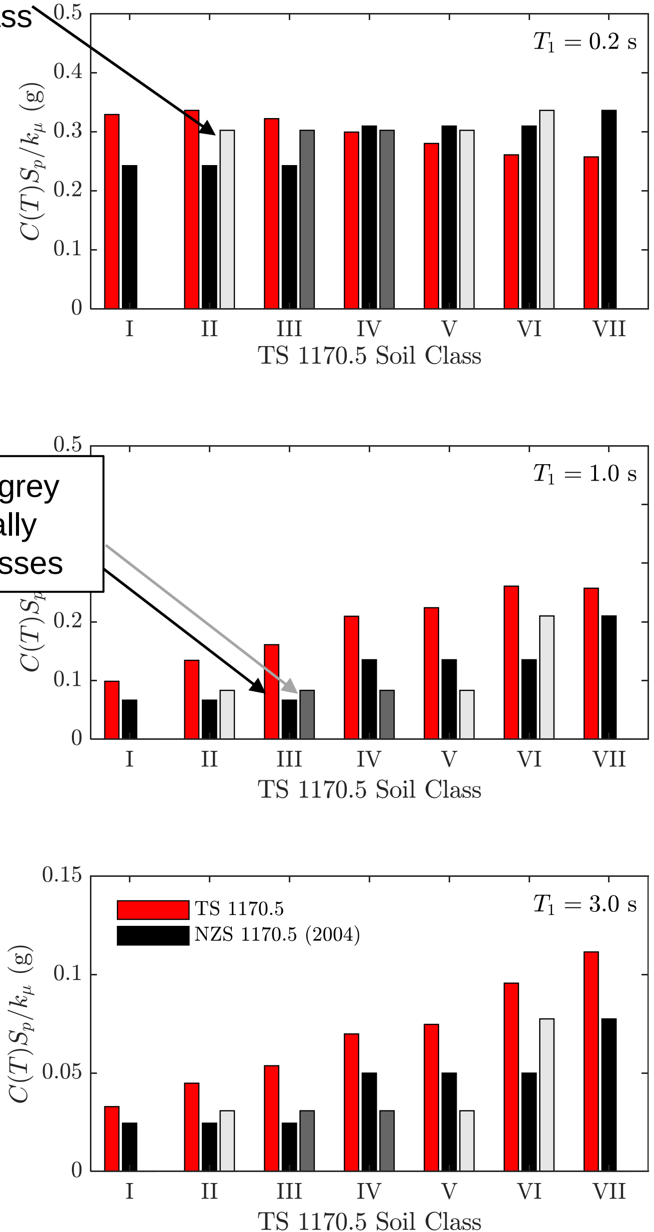
V_{s30} (m/s)	TS 1170.5 site class	NZS 1170.5 (2004)
150	VII	E
175	VI	D E
225	V	D C
275	IV	D C
375	III	B C
525	II	B C
750	I	A

WellingtonCBD, 500 years



Light grey lines and text show less relevant soil class

Black and dark grey lines show equally relevant soil classes



🟥 TS 1170.5

⬛ NZS 1170.5 (2004)

T = 0.2s

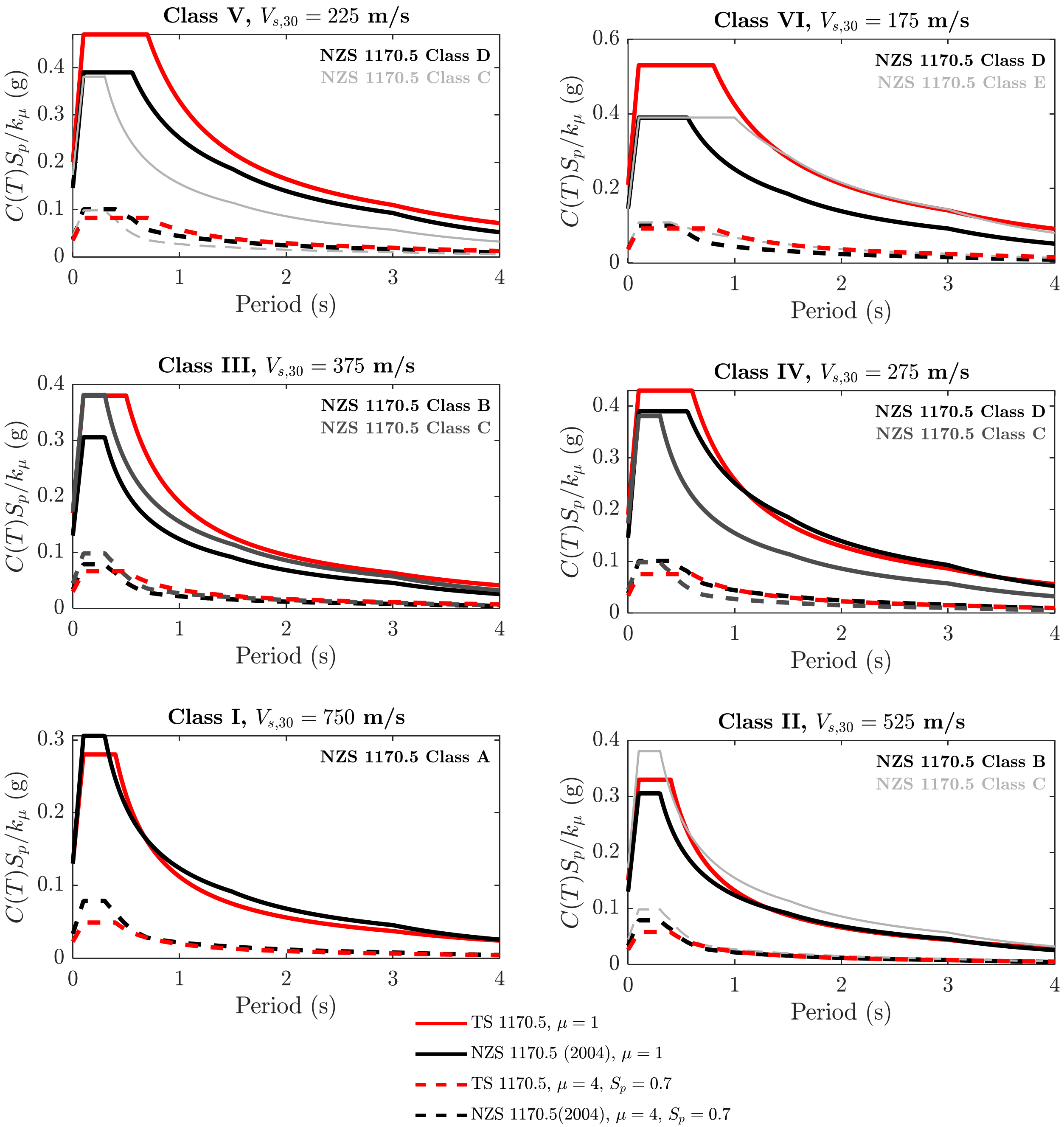
T = 1.0s

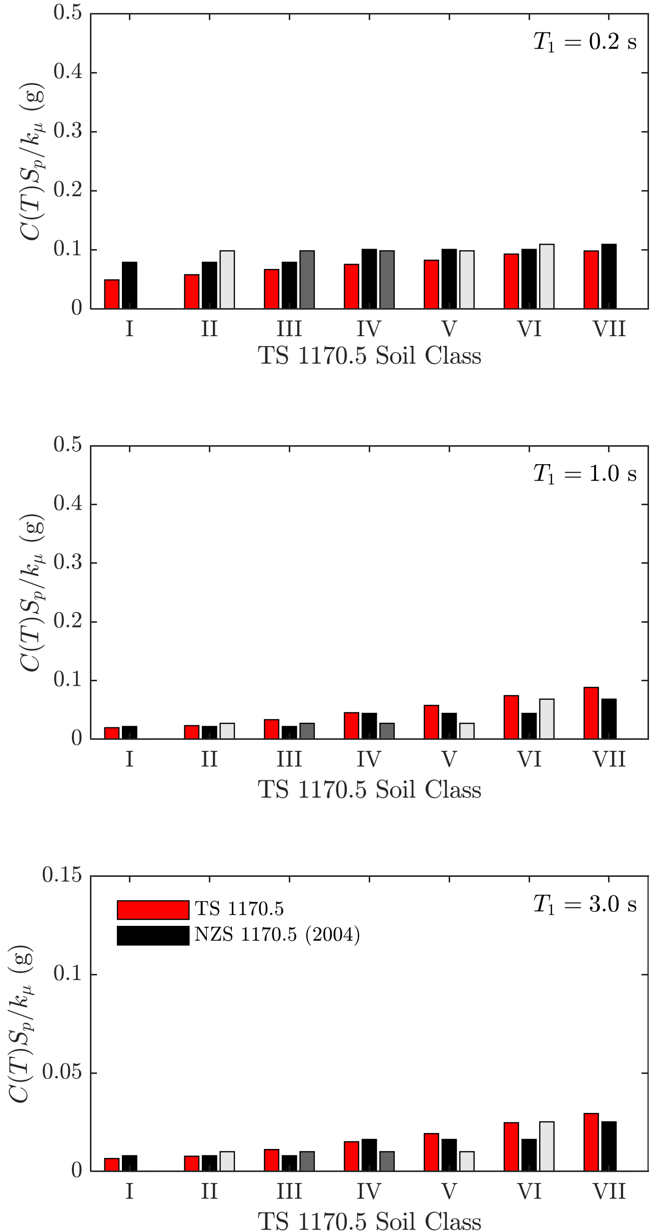
T = 3.0s

ROCK

SOFT SOIL

Auckland, 500 years





🟥 TS 1170.5
■ NZS 1170.5 (2004)

T = 0.2s

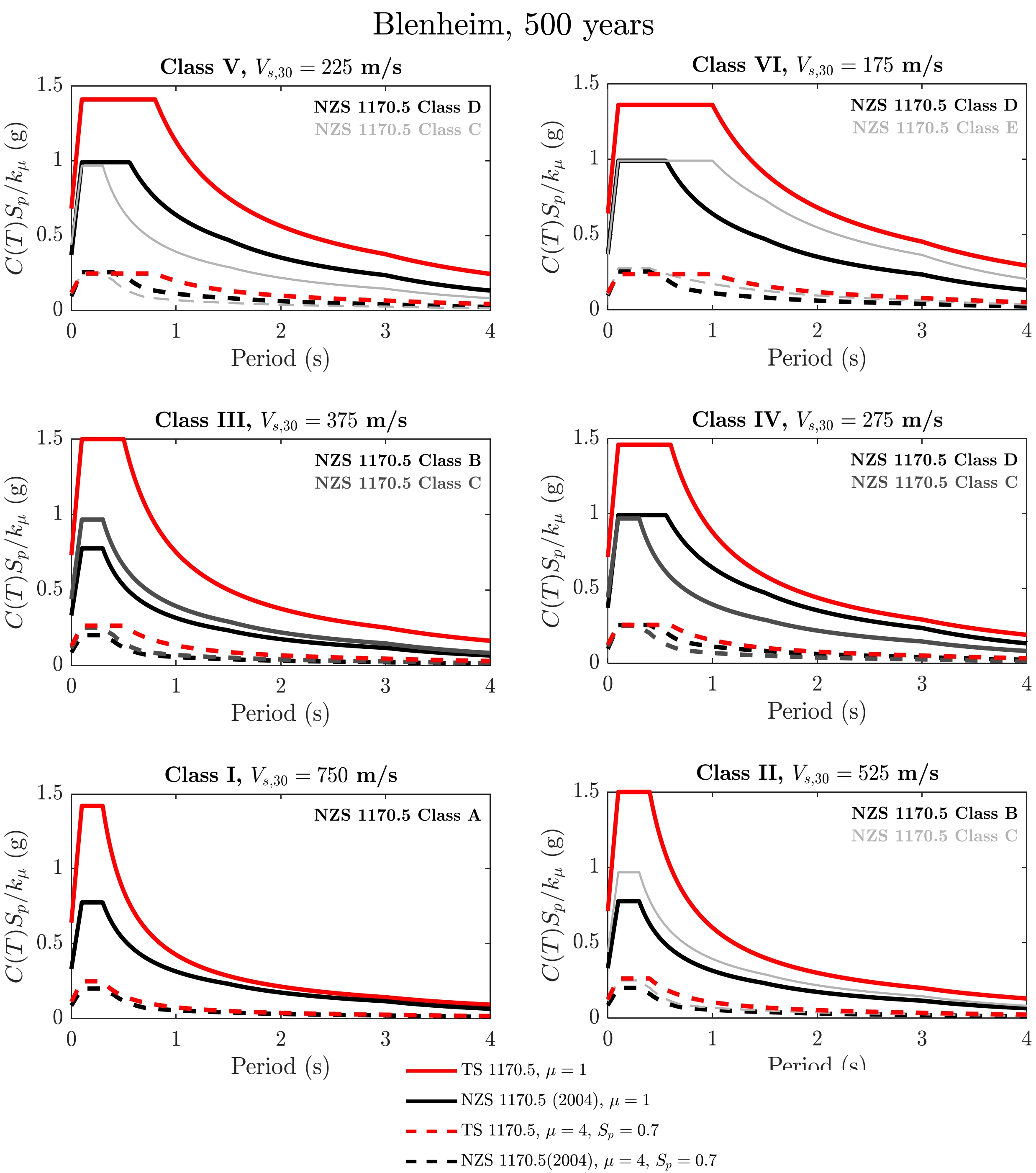
T = 1.0s

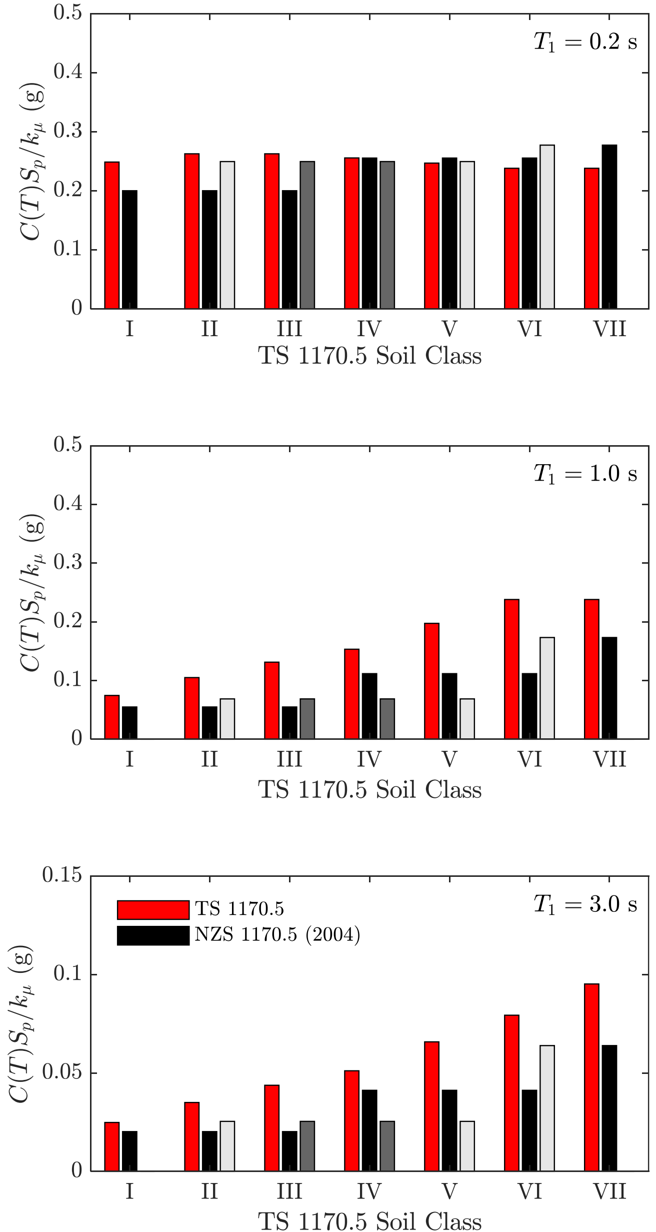
T = 3.0s

ROCK

SOFT SOIL

🟥 TS 1170.5
⬛ NZS 1170.5 (2004)





T = 0.2s

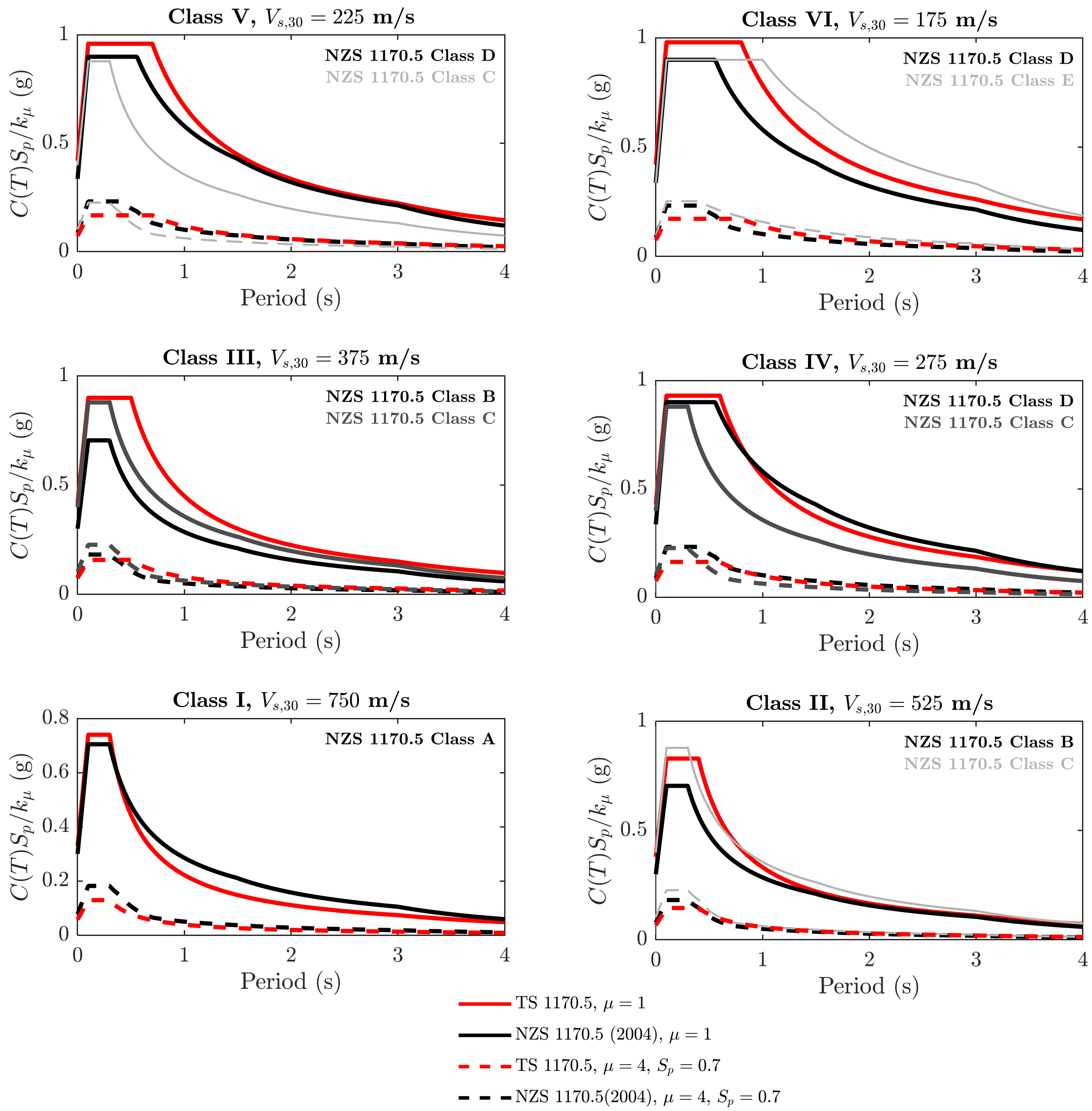
T = 1.0s

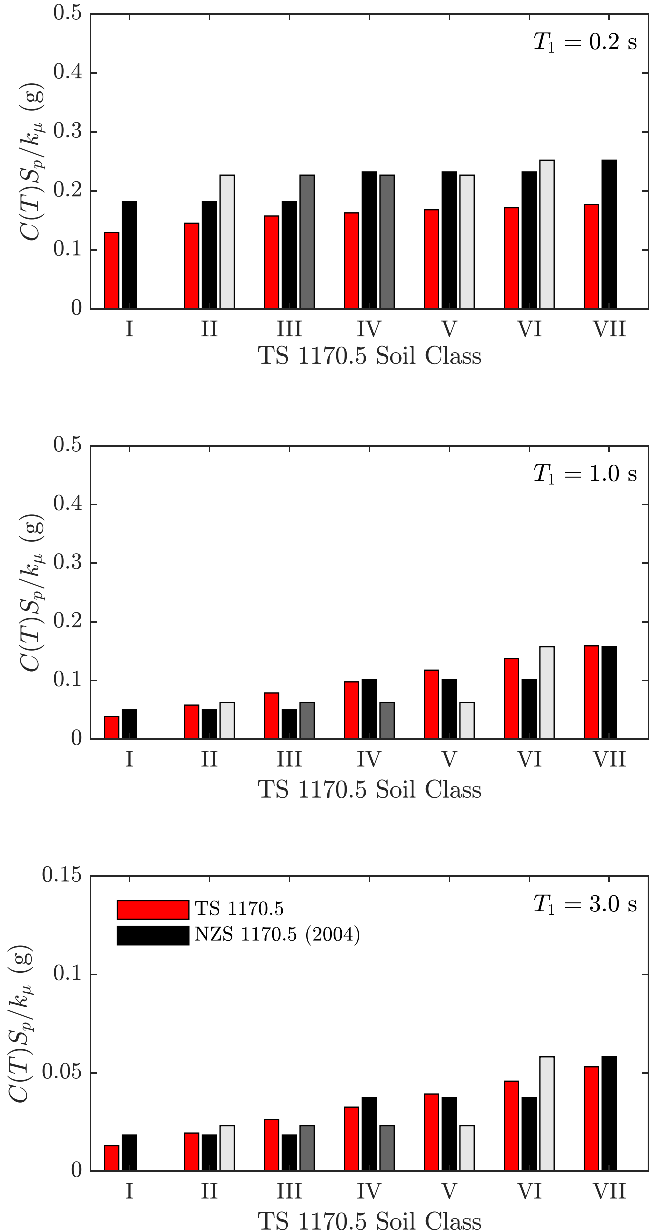
T = 3.0s

ROCK

SOFT SOIL

Christchurch, 500 years





🟥 TS 1170.5
⬛ NZS 1170.5 (2004)

T = 0.2s

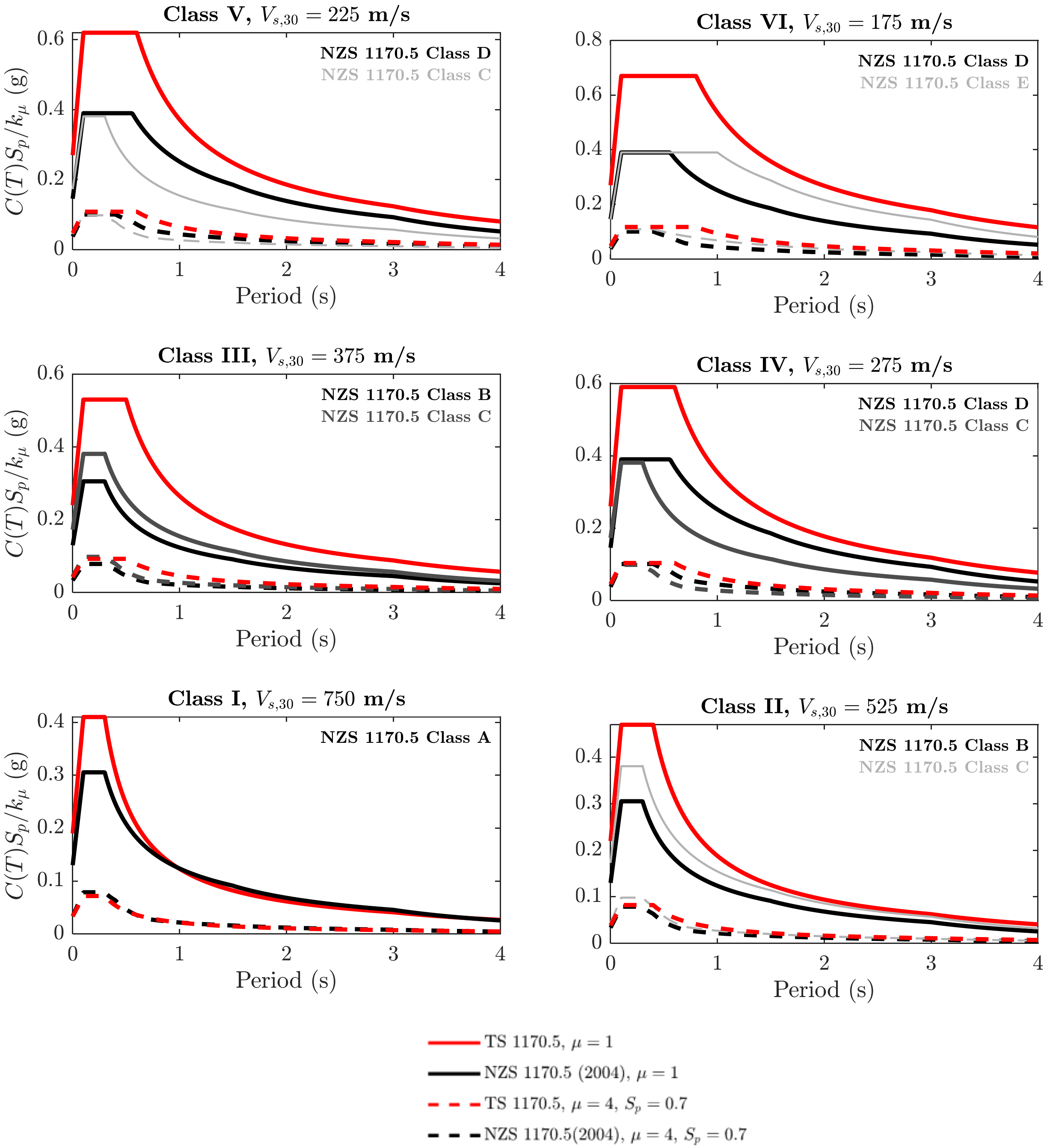
T = 1.0s

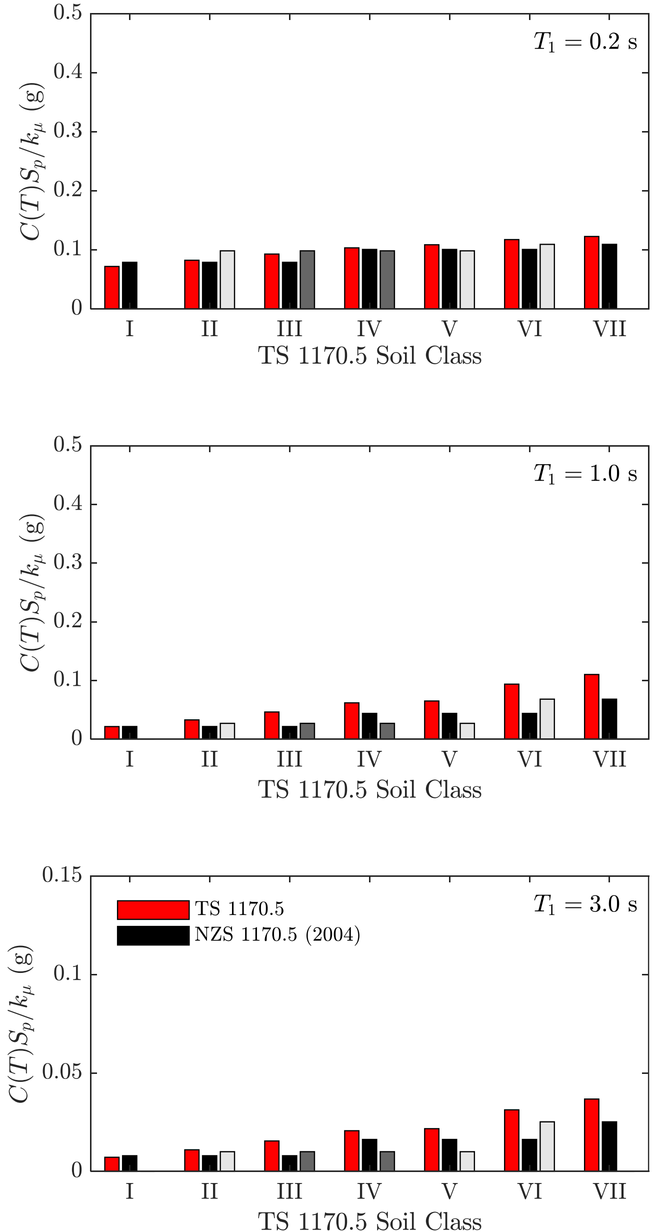
T = 3.0s

ROCK

SOFT SOIL

Dunedin, 500 years





🟥 TS 1170.5
⬛ NZS 1170.5 (2004)

T = 0.2s

T = 1.0s

T = 3.0s

ROCK

SOFT SOIL

Kia ora

Happy to answer questions at the end of today's webinar