

FURTHER COMMENTS ON SEISMIC DESIGN LOADS FOR BRIDGES

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SUMMARY:

This paper provides background material to the loadings section of the model code recently published by the Society's Discussion Group on Bridge Design, and presents a preliminary re-evaluation of the design spectra given in the proposed code. The basis for the proposed zoning scheme, in which the present uniform Zone B is replaced by a transition zone, is discussed. Arguments are given underlying the return period coefficients, and the force reduction factor used in generating the inelastic response spectra of the code. It is likely that the design spectra and the values of the other coefficients determining base shear forces will need to be revised as further research results become available; however, the form of the base shear expression, and the loadings section as a whole, should remain unchanged. Re-evaluated spectra suggest that the seismic coefficient values given in the proposed code may be too large by about 25 percent in Zone A, and too low by as much as 40 percent in Zone C. While the reassessed values should be more reliable than the original ones, they are based on a Japanese attenuation model, which has not yet been calibrated against New Zealand data. Further research is required to establish an appropriate attenuation model for New Zealand; to avoid undue proliferation of design loadings it is preferable to defer revision of the various coefficients in the proposed code until such a model is available. Until this is done, the proposed spectra should be viewed with caution, particularly in Zone C.

INTRODUCTION:

A recent edition of the Bulletin (Volume 13, No. 3, September 1980) carried the collected papers of a Discussion Group on Bridge Design, sponsored by the New Zealand National Society for Earthquake Engineering. Of the various topics covered by the discussion group, design ground motions was one of the more difficult because contemporary seismic risk analysis techniques had not been applied to New Zealand. The group was faced with the alternatives of mounting a major research program or of using standard, but less precise, techniques. In the event, the three-man loadings sub-committee was instructed by the group as a whole to produce results quickly, using simple methods. This was done, at the cost of some conservatism, in the expectation that the design spectra and zoning map would be updated as results of more detailed studies became available.

The proceedings of the discussion group were written in model code and commentary form which limited the amount of background material that could be presented. These notes are intended to present some of the arguments underlying the loadings section of the group's proceedings, "Design Loading and Ductility Demand"¹, and to present a preliminary reevaluation of the design spectra.

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SEISMIC BASE SHEAR FORCE:

The expression adopted for determining the design value of the horizontal seismic base shear force, H , was as follows:

$$H = C_{H\mu} Z_H M g \quad (1)$$

where

$C_{H\mu}$ = basic horizontal force coefficient

Z_H = return period coefficient

M = mass participating in lateral motion,

and

g = acceleration due to gravity

By introducing earthquake return period into the determination of design base shear force, the risk of the design value being exceeded (not necessarily equal to the risk of failure) is more obvious, and is given some perspective. Also, it allows the designer more freedom in selecting an appropriate degree of seismic resistance, and opens the way to cost-benefit studies and formulation of a reliability-based design code*. It was the intuitive feeling of a majority of the discussion group that most bridges should be designed for ground motions with a 150 year return period. Hence $C_{H\mu}$ and Z_H were normalized so that $Z_H = 1$ for a return period of 150 years.

*The discussion group considered seismic effects only. A probabilistic design code should also consider random variations in dead, live and other loads, as well as in structural strength, to arrive at risk factors related to the overall probability of failure. See, for example, Elms².

BASIC FORCE COEFFICIENT, $C_{H\mu}$

The horizontal force coefficient, $C_{H\mu}$, depends on the seismic zone, the fundamental natural period of vibration of the structure, and on the design value of the displacement ductility factor, μ . For $\mu = 1$, $C_{H\mu}$ estimates elastic, absolute acceleration response, for 5 percent of critical damping, with a return period of 150 years. Estimates of inelastic response are obtained by multiplying elastic response values by a force reduction factor. The derivation of the elastic response spectra, the seismic zones and the force reduction factor are discussed below.

Seismic Zones -

The proposed zoning scheme, shown in Figure 1, does not differ greatly from that of the current New Zealand design code, NZS 4203³. Three zones are retained. The main change is to Zone B, which in the proposed scheme provides a smooth transition between the two zones of assumed uniform seismicity, A and C, reflecting seismic risk more accurately than in the present scheme.

The map shown in Figure 1 is based principally on Smith's⁴ contour maps of Modified Mercalli intensities for given return periods, reflecting seismicity observed during European settlement, and on Lensen's "Late Quaternary Tectonic Map"⁵, indicating regions of active faulting in geologically recent time. In some areas, these sources are in conflict; this is not surprising, since they result from very different methods, and observe short and long time periods respectively.

One area where recorded seismicity seems lower than expected from geologic evidence is in the Alpine Fault region of the South Island. On the basis of Smith's intensity contours, this area should be in Zone B. However, it would clearly be imprudent to exclude the principal fault in the country from Zone A. This conclusion is reinforced by Adam's recent evidence⁶ for the occurrence of great earthquakes on the Alpine Fault at intervals of about 500 years in the past.

Taranaki is another difficult region. High historical seismicity in this area is reflected in Smith's maps, which suggest that the Zone A - Zone B boundary should lie in an east-west direction. But the main tectonic features of the North Island lie in a north east - south west direction. These features comprise the Hikurangi trench, and the main mountain ranges and fault systems⁷. They indicate that the zone boundary, as well as the Zone B risk contours, should lie parallel to the east coast of the North Island. However, there is other geologic evidence to support a more east-west trending boundary. Earthquakes occurring in the region of crustal tension north of Taupo, indicated by the thermal area extending into the Bay of Plenty and by the Hauraki grabben (Lensen, personal communication), should be

less energetic than those occurring further south at similar distances from the main fault system. Also, the soft crustal material of the thermal region should shield the Waikato to some extent from seismic waves originating in the main tectonic region. Hence we should expect seismic risk in Taranaki to be higher than in Waikato.

Other information used in determining the boundaries and contours in Figure 1 were Matuschka's⁸ contour maps of intensities and of peak ground accelerations and velocities for selected return periods, a map of epicenters of large earthquakes, shown in Figure 2, and a map of density of energy release, shown in Figure 3. The latter two maps were prepared from seismological data recorded from 1840 (Smith⁹).

These diverse and not particularly consistent sources of information suggested that a zone of uniformly high risk should be retained in roughly the same region as Zone A of the present standard. Also, to provide a minimum level of seismic resistance, a uniform Zone of low risk should also be kept in the present North Island Zone C. Elastic response spectra for Zones A and C were computed from Smith's⁴ 150 year M.M. intensities for Wellington and Auckland respectively. These locations were chosen because they are in the more active parts of their respective zones, and also because they are the locations likely to have the greatest amounts of construction. The zone boundaries and contours were then fixed, quite subjectively, by sketching with the above points in mind and with the various maps in view.

Elastic Response Spectra -

The elastic acceleration response spectra from which the base shear force coefficients were obtained were computed from Smith's 150 year return-period M.M. intensities. First, peak ground accelerations and velocities were computed from intensities, using the correlations of Trifunac and Brady¹⁰ and of Murphy and O'Brien¹¹. These were found to agree fairly closely with each other, and to be consistent with values estimated independently by Matuschka⁸. Five percent damped elastic response spectra were then computed from the peak motion values, again using two methods, that of Seed et al¹², and that of Newmark and Hall¹³ and Mohraz¹⁴. Again, the spectra found by the two methods agreed quite closely. In Zone A the spectrum derived by Seed's procedure, for deep cohesionless soil deposits, was used. In Zone C Mohraz's method was used, but with a higher peak ground velocity than expected from the 150 year Auckland intensity, to allow, at least qualitatively, for the effect of distant earthquakes.

The Zone B spectrum has the average shape of the Zones A and C spectra, and is normalized so that the steps in spectral ordinates that occur in passing out of Zone B are minimized, to a maximum value of about 15 percent.

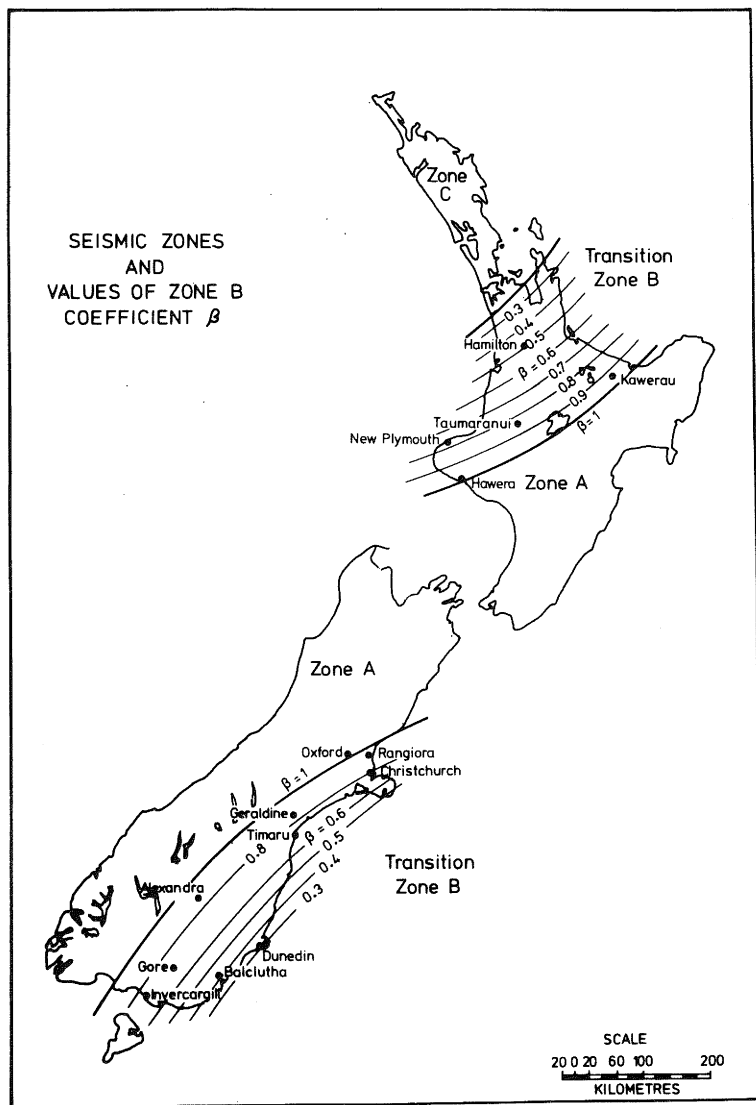


Figure 1 – Proposed Seismic Zones.

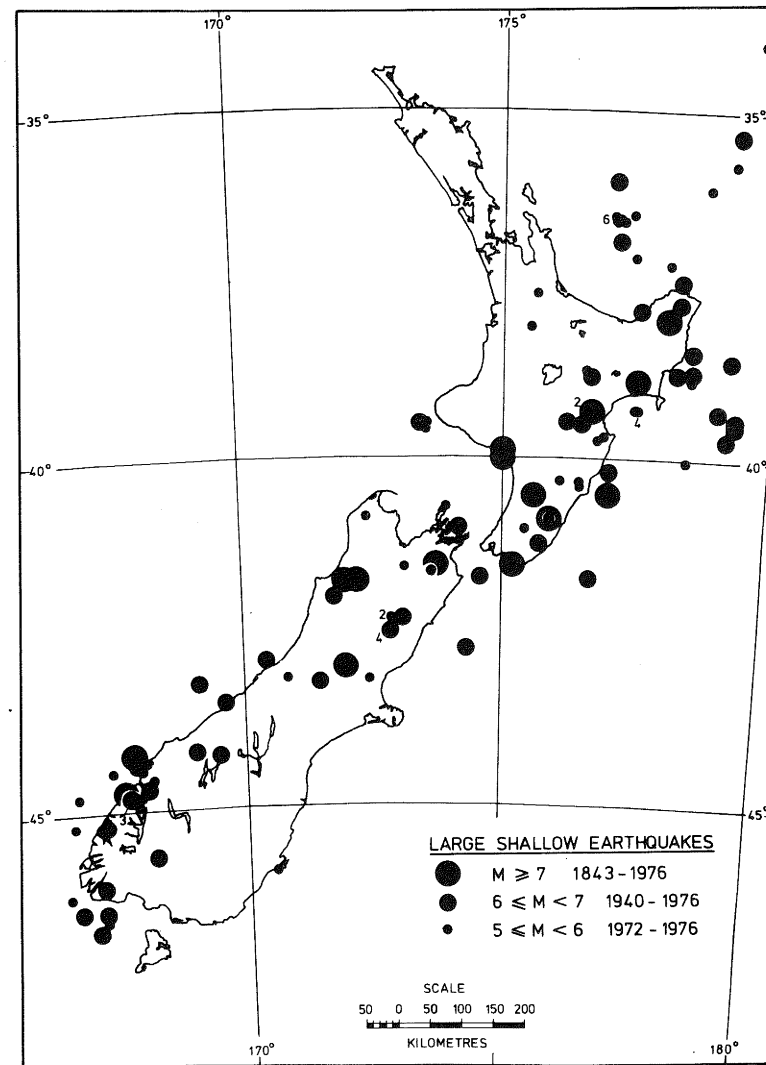


Figure 2 – Distribution of Earthquake Epicentres in New Zealand, Note that Shorter Sampling Periods have been Used for Smaller Magnitudes.

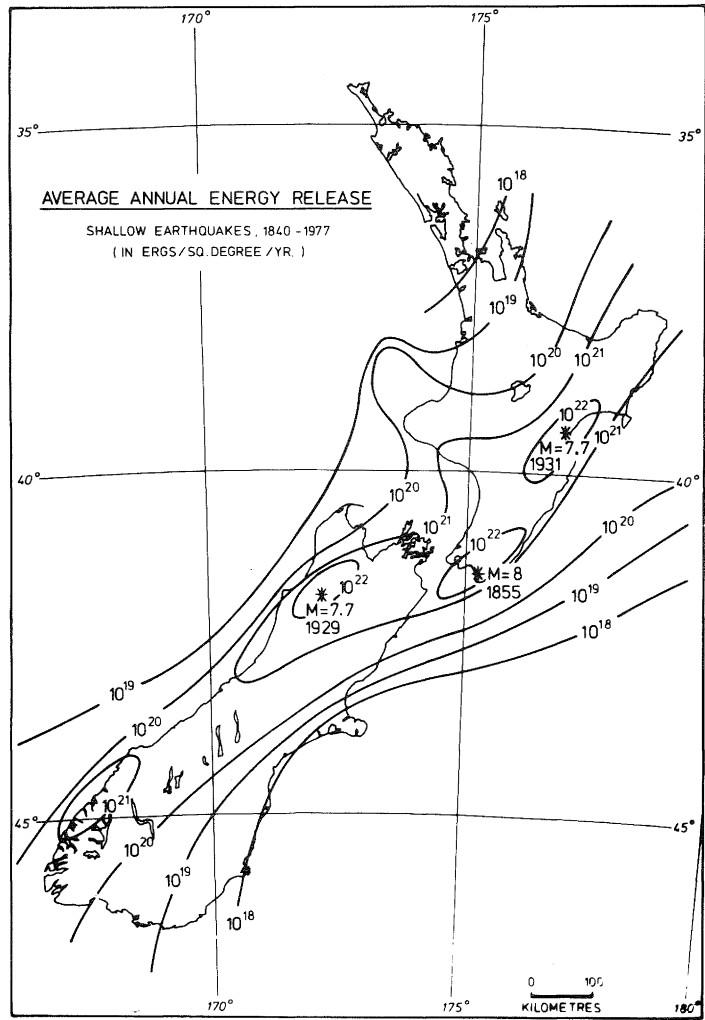


Figure 3 – Map Showing Density of Seismic Energy Release.

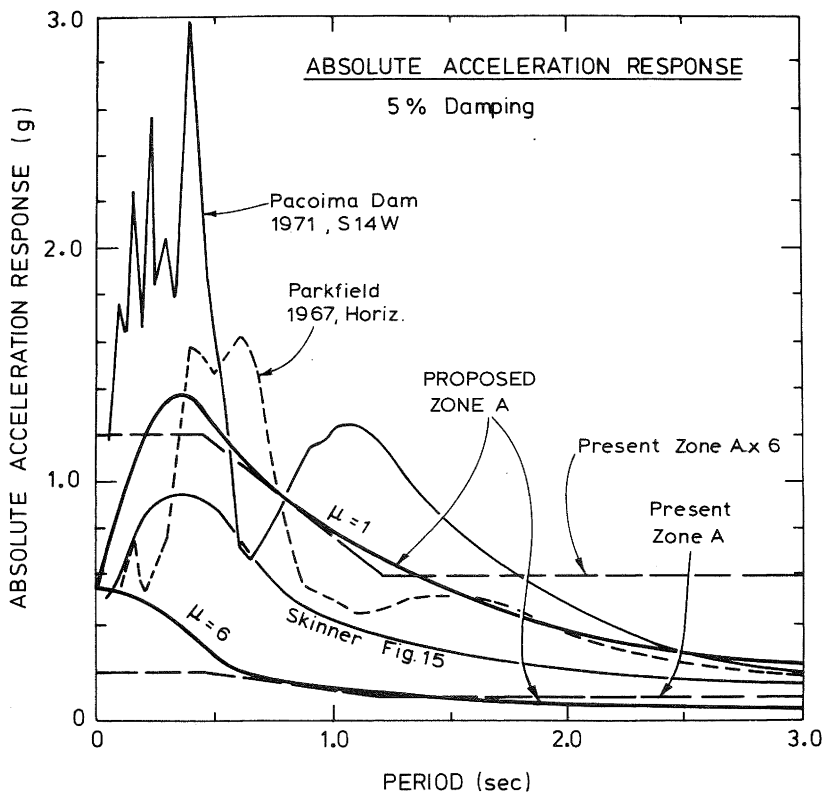


Figure 4 – Proposed Zone A Acceleration Response Spectra, Compared with Present M.W.D. Bridge Design Spectra and with Various Recorded Strong Motions. Note that Elastic Acceleration Response (with $\mu=1$) Corresponds to $CH\mu$ with $\mu=1$.

In Figure 4, the proposed Zone A design spectra (or values of C_{Hu}) are compared with corresponding Ministry of Works and Development design spectrum for bridges¹⁵ and with some spectra of well-known accelerograms. The proposed elastic spectrum should be compared with six times the M.W.D. curve since the accompanying design procedure is based on a ductility factor of six. It can be seen that the two curves agree closely in the middle period band. At short periods the proposed curves result in much higher design loads for ductile structures than given by the existing spectra¹⁵. At longer periods, the two curves separate due to the artificially horizontal plateau of the present spectrum (a constant acceleration response line implies unbounded relative displacement response, which is not possible).

Note that the proposed Zone A elastic spectrum is similar to that of the 1966 Parkfield, California accelerogram, recorded close to the source of a small, $M=5.6$, earthquake. About four such earthquakes occur per year in New Zealand.

Another point to note from Figure 4 is the divergence of the proposed, $\mu = 6$ spectrum from the present spectrum at periods smaller than about 0.6 seconds. This occurs because in obtaining the present curves, a constant force reduction factor, proportional to $1/\mu$, was applied to Skinner's¹⁶ average spectrum (also shown in Figure 4). However, as explained below, use of a constant reduction factor is not valid at periods smaller than about 0.7 seconds; because of this, the present spectra are unconservative at small periods.

A similar comparison is made in Figure 5 for the Zone C spectra. Here, the proposed values of C_{Hu} are much smaller than the present ones. They are also smaller than those from a more rigorous, but preliminary, re-evaluation of the design spectra which is discussed further below.

Return Period Coefficient Z_H -

Values of Z_H (Table 2.1) are proportional to peak ground velocities, computed from Smith's⁴ M.M. intensities for various return periods. Trifunac and Brady's¹⁰ relation between peak velocity and intensity was used. Originally, separate values of Z_H were computed for each zone, but they were so close that a single set of values was used in the final version of Table 2.1. Peak ground velocity was used for scaling because it characterizes spectral strength at around 1 second, and is therefore more representative of the strength of the spectrum as a whole than, say, peak acceleration.

Site Conditions -

Site soil conditions are not required to be considered in obtaining values of the seismic coefficient C_{Hu} from the proposed code. The single set C_{Hu} of curves giving values of C_{Hu} was estimated for deep alluvial soils, typical of many New Zealand bridge sites. It was felt that techniques

for predicting "site effects" were not well enough advanced for precise predictions of local effects to be made reliably. Two exceptions are mentioned in the commentary. They are hard rock sites, where it is recommended that values of C_{Hu} be increased at periods less than 0.4 seconds, and sites with very soft, uniform surficial deposits. The latter is the only case in which it was considered that one-dimensional "shear-beam" analyses could be employed usefully. The problem of assessing site effects, together with other sources of uncertainty in estimating strong ground motions, is discussed briefly in reference 24.

Force Reduction Factor -

As mentioned above, the existing loadings code for buildings (NZS 4203)³ and also the Highway Bridge Design Brief²¹ have seismic coefficients established by reducing an assumed elastic response spectrum by a constant multiplier, R , equal to the inverse of the displacement ductility factor. That is,

$$R = \frac{1}{\mu} \quad (2)$$

This practice is based on the equal displacement principle, which has been shown by numerous inelastic time-history analyses of simple oscillators to give a reasonable estimate of the response of structures with periods longer than about 0.7s. It has, however, long been recognised that for short period structures, the approach is non-conservative, and could result in excessive ductility demand.

The equal energy principle has been proposed as being applicable for short to medium period structures, and results in a force reduction factor of

$$R = \frac{1}{\sqrt{2\mu - 1}} \quad (3)$$

Thus for a displacement ductility factor of $\mu = 6$, the equal energy principle results in a reduction factor of 0.302, compared with 0.17 implied by the equal displacement principle.

For very short period structures (say $T < 0.2s$) the force reduction factor given by Eq. 3 has been found to be still unconservative. Gulkan and Sozen²² quote displacement ductility factors of 28 to 30 resulting from a $T = 0.15s$ structure designed for a force reduction factor of about 0.3, and analysed under different earthquake records. The equal displacement and equal energy principles would result in expected ductility demand of 3.33 and 5.95 respectively. This inadequacy of the equal energy principle for short period structures results from a tendency for the period to lengthen and degrade into a period-range of higher response, as a result of inelastic action²³. For medium and long period structures, the period lengthening causes a shift away from the period range of maximum response.

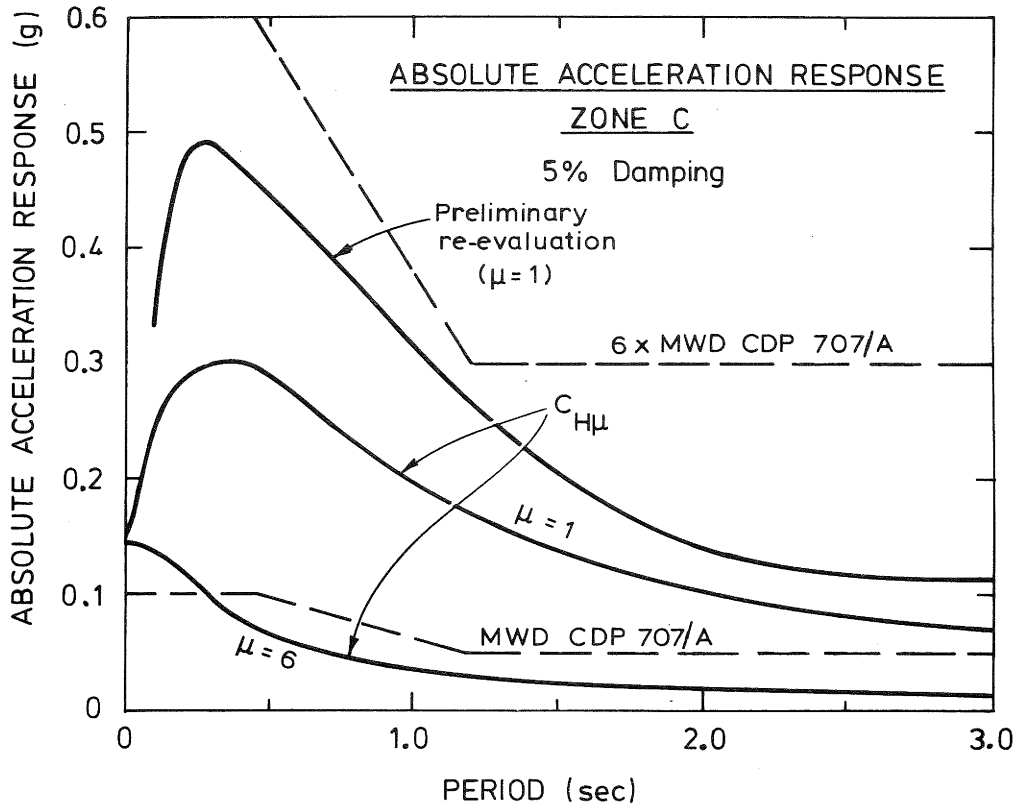


Figure 5 – Comparison Between Proposed Values of Seismic Coefficient $C_{H\mu}$ for Zone C, and Those Obtained in a Preliminary Re-evaluation Using more Rigorous Seismic Risk Estimation Techniques.

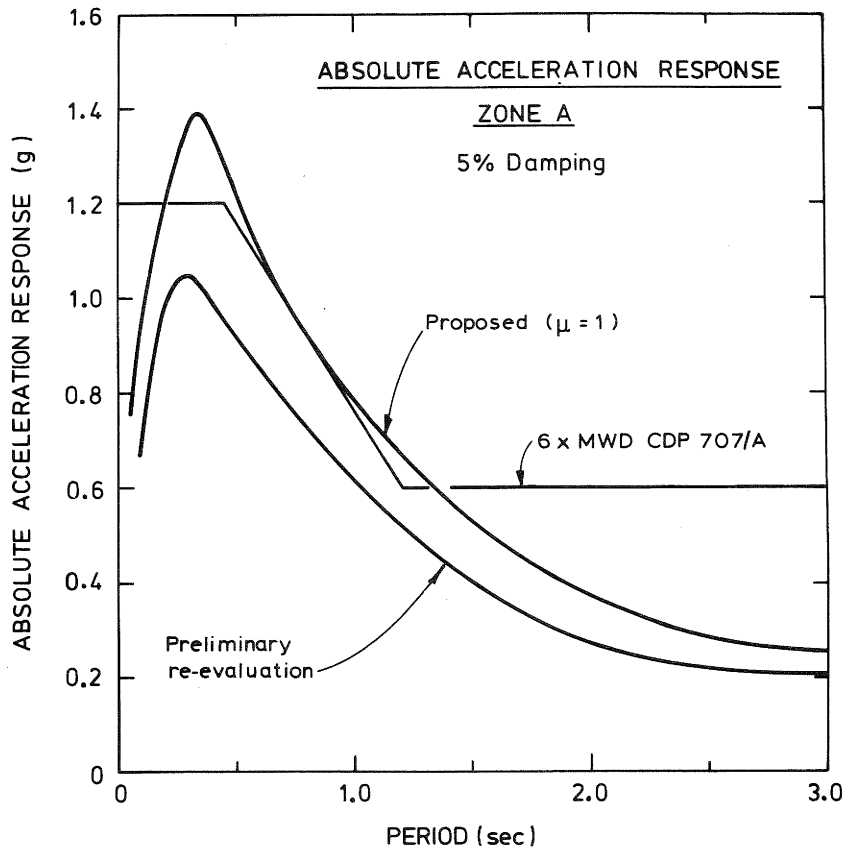


Figure 6 – Comparison Between Zone A elastic Acceleration Response ($C_{H\mu}$ for $\mu=1$) in Proposed Code, and Corresponding Spectrum Obtained in Preliminary Re-evaluation using Peek's Seismicity Model and Katayama's Attenuation Model.

In the limit, when the period approaches $T = 0$, the structural response acceleration will be equal to the peak ground acceleration, regardless of the ductility factor. Consequently the force reduction factor should be $R = 1.0$ at $T = 0$.

The approach adopted in the loadings section of the proposed bridge design code was to reflect the above trends by accepting the equal displacement principle for $T > 0.7s$, use the equal acceleration principle for $T = 0$, and to specify a linear variation between these methods for intermediate periods. Thus the force reduction factors are for $T = 0$.

$$\text{for } T = 0 \quad : \quad R = 1.0 \quad (4a)$$

$$\text{for } 0 < T < 0.7s \quad : \quad R = \frac{0.70}{(\mu-1)T+0.7} \quad (4b)$$

$$\text{for } T \geq 0.7s \quad : \quad R = \frac{1}{\mu} \quad (4c)$$

For $\mu = 6$, Eq. 4b gives the same force reduction factor as the equal energy principle for a period of $T = 0.32s$. At lower periods, it is more conservative. Although Eq. 4b is more the result of intuition than extensive calibration, it thus appears to provide a reasonable estimate of response throughout the period range.

PRELIMINARY RE-EVALUATION OF SPECTRA:

Application of probabilistic seismic risk analysis techniques, pioneered by Cornell¹⁷, to New Zealand has been studied by Peek¹⁸. He has established a seismicity model for the country, based on seismological, geologic and tectonic data. This is one of the two principal components of a seismic risk analysis, and describes the distribution of earthquake magnitudes as a function of geographical position. The other component is an attenuation expression, relating magnitude to strength of shaking at a site, through epicentral or focal distance. Peek examined several attenuation expressions available in the literature, and found that none was immediately applicable to New Zealand. Because of the high sensitivity of predicted design, or risk spectra to details of the attenuation expression, worldwide and New Zealand data are being studied in an attempt to derive an attenuation model more representative of New Zealand geologic conditions, and it is questionable whether the spectra presented in the proposed code should be revised before results of this study are available.

However, preliminary estimates of 150-year response spectra have been made using an attenuation expression derived from Japanese strong motion accelerograms by Katayama *et al.*¹⁹ The Japanese expression is favoured because of the tectonic similarities between Japan and New Zealand.

In computing the spectra for Auckland and Wellington, shown in Figures 5 and 6 respectively, some judgement has been

exercised in selecting values of the scatter parameter, σ_{10} in Peek's notation¹⁸.

This parameter measures the spread of spectral ordinates, treated as random variables, about the mean attenuation curve. (The greater the scatter, the greater the probability of a given strength of ground motion; hence the greater the spectral ordinates for a given return period¹⁸.) Katayama *et al.* suggest a value of $\sigma_{10} = 0.3$

at all periods, and this would be the appropriate value to use for sites in Japan. However, unless New Zealand conditions are identical to those of Japan, a greater value of σ_{10} is required. Guided by Trifunac and Anderson²⁰, values of σ_{10} increasing from 0.3 at a natural period of 0.2 seconds to 0.48 at 3 seconds have been used.

Figures 5 and 6 suggest that the values of $C_{H\mu}$ given in the draft bridge code¹ are overestimated in Zone A by about 25 to 30 percent, and under-estimated in Zone C by about 40 percent. However, the spectral shapes are similar; bearing in mind the uncertainties involved in both methods of computations, the differences are not unexpected nor particularly great. However, they suggest that if use is to be made of the draft code in Zones B and C before a proper revaluation has been carried out, higher values of $C_{H\mu}$ should be used. Above all, the comparisons point up the importance of the attenuation study in progress.

Re-evaluation of the return period coefficient Z_H yields the results shown in Figure 7. These suggest that the values given in Table 2.1 of the proposed code¹ are too large for return periods greater than 150 years and too small for shorter return periods. The close agreement between the Auckland and Wellington values confirms the use of a single set of coefficients for all zones. As with the spectra themselves, values of Z_H depend on the attenuation expression used; again, it is doubtful whether the values of Z_H in the proposed code should be revised before the attenuation study in progress is completed.

CONCLUSIONS:

The design spectra presented in the recent paper on Bridge Earthquake loadings¹, and described further in this paper, represent a first attempt to provide a more rational basis for seismic design loading for New Zealand bridges than was previously available. Although the data for establishing the spectra are sparse, and seismic risk models for New Zealand are still in preliminary stages of development, the methodology presented is felt to be sound. Consequently, improvements in seismicity models and attenuation expressions are expected to result in (hopefully small) changes to the basic response spectra, but should not affect the method of calculating design loads. Already recent work has indicated that the response ordinates for

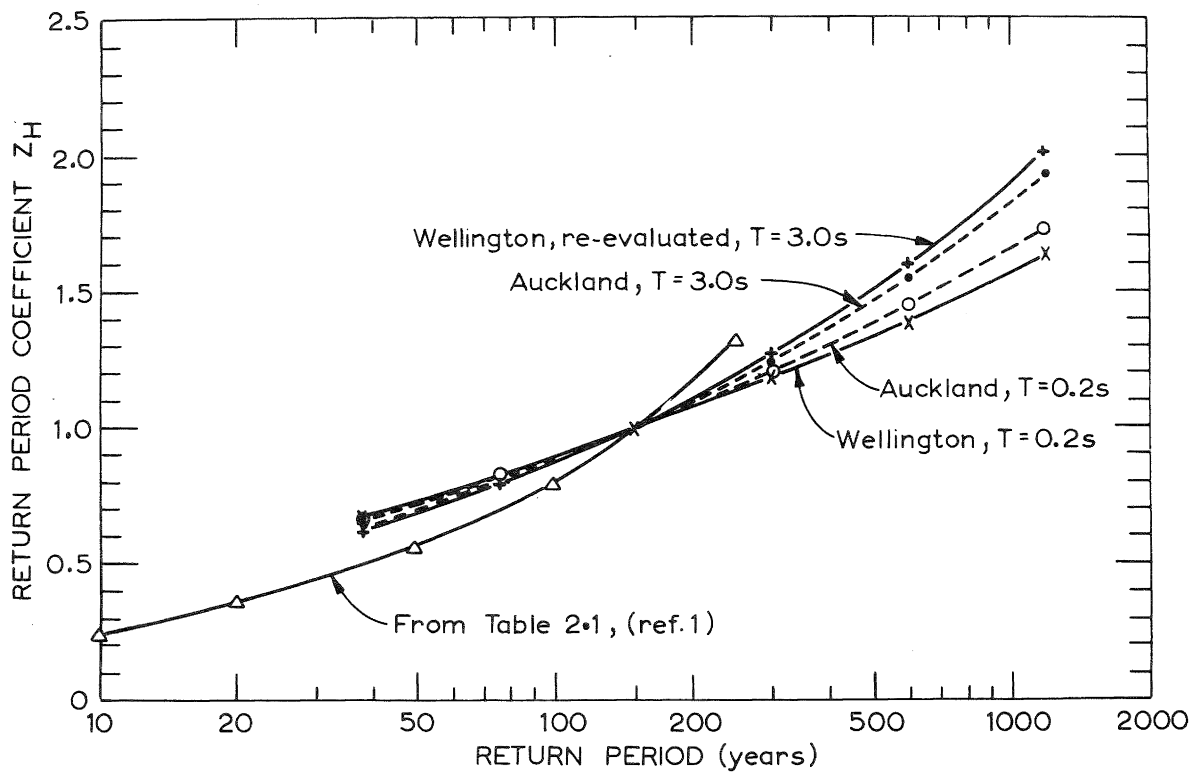


Figure 7 – Values of Return Period Coefficient Z_H Obtained in Preliminary Re-evaluation, at Natural Periods of 0.2 and 3.0 sec, for Auckland and Wellington Sites. Values from Table 2.1 of Proposed Code¹, are also shown.

Zone C may be low by as much as 40 percent. It is expected that continuing research will result in revised spectra that can be adopted with an acceptable degree of confidence within the next few years. Until such time, the curves should be viewed with some caution, particularly for Zone C, though it is felt that they represent a greatly improved option to the existing spectra incorporated in New Zealand bridge and building codes.

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