SEISMIC DESIGN SPECTRA FOR DIFFERENT SOIL CLASSES

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SUMMARY

This paper investigates the validity of the soil class dependent spectral shape factors used to calculate seismic design actions in the New Zealand seismic design standard NZS1170.5, which currently specifies seismic design spectra corresponding to five different soil classes. According to the current provisions stipulated in NZS1170.5, for all natural periods, the seismic demand for structures on soft soil is either equal to or greater than that for structures on hard soil. This is opposite to the basic structural dynamics theory which suggests that an increase in stiffness of a system results in an increase in the acceleration response. In this pretext, a numerical parametric study is undertaken using a nonlinear site response analysis tool in order to capture the effect of soil characteristics on structural seismic demand and to scrutinize the validity of the current site specific seismic design spectra. It is identified that the level of input ground motion intensity and shear stiffness of the soil deposit (represented by its shear wave velocity V_s) greatly affect the maximum acceleration and frequency content of the surface motion. The study found some shortfalls in the way the current code defines seismic design demand, in particular the hierarchy of soil stiffness at low structural periods. It was found that stiff soils generally tend to have a higher spectral acceleration response in comparison to soft soils although this trend is less prominent for high intensity bed rock motions. It was also found that for medium to hard soils the spectral acceleration response at short period is grossly underestimated by the current NZS1170.5 provisions. Based on the outcomes of the parametric numerical analyses, a revised strategy to determine structural seismic demand for different soil classes is proposed and its application is demonstrated through an example.

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INTRODUCTION

Among the different analysis methods (e.g. linear static, nonlinear static, linear dynamic and nonlinear dynamic analysis methods) used to determine seismic design actions in building components, the equivalent static analysis procedure, in which a lateral force is calculated and then applied to the structure as a set of equivalent static forces, is widely accepted in seismic codes of most earthquake prone countries. In this approach, the lateral design seismic force is calculated as a product of the building weight and a seismic design coefficient, which depends on the building period, use (i.e. importance factor or its equivalent), inherent ductility, local seismicity and local soil characteristics.

In NZ, the Australia/New Zealand Standard: Structural design actions Part 5: Earthquake actions - New Zealand, commonly referred as NZS1170.5 [SNZ 2004], is used to determine seismic design demand, which is calculated in the form of shear force at the base of the structure being designed. As shown in Equation (1a) below, the design base shear V_b is calculated as the product of the seismic weight of the structure and a seismic design coefficient $C_d(T)$ which caters for inelastic design by reducing the elastic strength demand in proportion to the available ductility. The elastic demand is expressed in terms of a coefficient $C(T)$ known as the elastic site hazard spectrum, which is calculated as shown in Equation (1c) below.

$$V_b = C_d(T)*W \tag{1a}$$

$$C_d(T)=C(T)*S_P/K_\mu \tag{1b}$$

$$C(T) = C_h(T)*Z*R*N(T, D) \tag{1c}$$

where S_P = structural performance factor

K_μ = ductility factor

$C_h(T)$ = the spectral shape factor

Z = the zone factor

R = the return period factor

N = the near-fault factor

In this approach, the local site effect is accounted for via the spectral shape factor $C_h(T)$ (see Figure 1) which characterises the seismic demand corresponding to five different soil classes (alphabetically categorised from A/B: rock to E: soft soil). The $C_h(T)$ factors are the spectral acceleration response values at the corresponding periods obtained from hazard analyses normalised with respect to the local seismicity [McVerry 2003], as shown in Equation (2).

$$C_h(T) = \frac{S_a(T)}{Z} \tag{2}$$

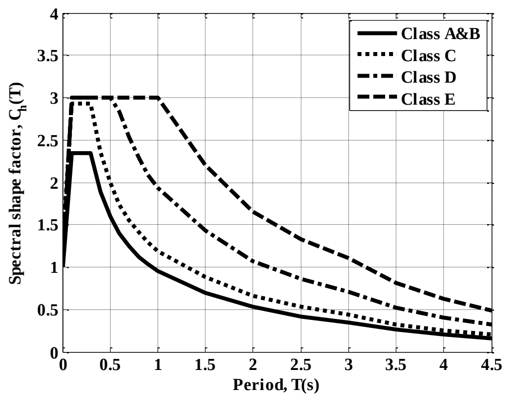

Figure 1: ***Spectral shape factor in NZS1170.5.***

In Equation (2), the zone factor Z represents the local seismicity, and is determined as half of the spectral acceleration at 0.5 sec (i.e. $S_a(0.5s)$) for a shallow soil condition (i.e. soil type C). For example, Z values are equal to 0.13, 0.3, and 0.4 for Auckland, Christchurch (after the Canterbury earthquakes [McVerry *et al.* 2012, Royal Commission 2012]), and Wellington, respectively. It is worth mentioning that the spectral shape factor is independent of the location (i.e. local seismic intensity represented by the zone factor Z) in the current version of NZS1170.5. In other words, the $C_h(T)$ curve (Figure 1) to be used for Auckland (low seismic zone) is exactly the same as that for Wellington (high seismic zone).

Local site effect – i.e. "seismic motions at the surface of a soil deposit can have significantly different characteristics from motions at the underlying bedrock and different types of soil deposits modify the bedrock motions differently" – has been extensively observed in previous earthquakes; e.g. Loma Prieta 1989 [Seed *et al.* 1990] and Mexico City 1985 [Dobry and Vucetic 1987]. Depending on the depth, shear modulus and plasticity of the soil deposit as well as the intensity, frequency content and duration of the bedrock motions, the seismic motions can be amplified or de-amplified at the ground surface. The local site effect is acknowledged universally in most seismic design codes, but different codes account for this effect differently.

As shown in Figure 1, NZS1170.5 currently considers a hard to soft soil hierarchy in terms of expected spectral acceleration response. In other words, the specified spectral shape factor reduces as the soil gets harder. For any value of natural period, the elastic design demand for a soft soil is either equal to or greater than (more than three times at some periods) that for a harder soil. However, this is in contrast with the basic structural dynamic principle that stiffer systems attract greater force.

The origin of the notion that soft soils amplify earthquake motions travelling from the bedrock underneath, which appears to be the basis of the local site effect consideration currently adopted in most seismic codes (including NZS1170.5) can be tracked to some reported evidences observed in the previous earthquakes; especially the Mexico City earthquake. Nevertheless, there are several evidences which also indicate higher amplification in a rock than on a soil site. One such evidence is the statistical study [Seed *et al.* 1976] using 147 records from the western USA, as shown in Figure 2. Most recently, the acceleration response spectra (see Figure 3) of the ground motion recorded on rock (LPCC) and soil (LPOC) in Lyttelton during the 22 February earthquake [Cubrinovski and McCahon 2011] showed again that acceleration amplification is higher on the rock site; especially in the short period range.

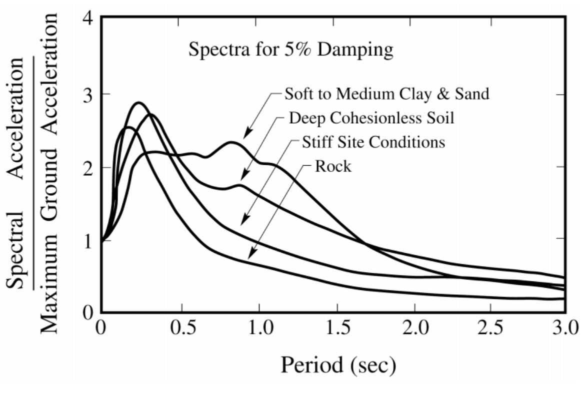

Figure 2: ***Average acceleration spectra for different site conditions [Seed et al. 1976].***

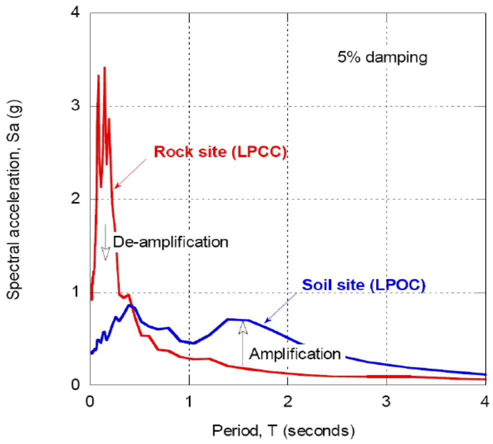

***Figure 3:** Acceleration response spectra during the 22 February 2011 Christchurch earthquake [Cubrinovski and McCahon 2011].*

In order to evaluate the validity of the current code provisions on local site effect, the effect of sub-soil properties on the characteristics of ground motions transferred to the surface and the resulting structural demand are investigated in this study through a numerical parametric analysis. Comparison is made to determine how well the current consideration represents the response of different soil classes to seismic excitation. Furthermore, it also provides recommendations for possible revisions to the elastic design spectrum currently being used in the force-based seismic design procedures.

NUMERICAL MODELING AND ANALYSIS TOOL

To quantitatively capture the structural demand due to different soil classes, series of numerical analyses with varying soil properties and input bedrock motions are conducted. In this section, the classification and characterization of different soil classes are discussed first, followed by the illustration of the numerical model as well as the input motions.

Soil classes

Five site subsoil classes are defined in NZS1170.5. The classes are alphabetically categorized, and are based on the site period. Four different methods of calculating the site period are detailed in Clause 3.1.3.1 of NZS1170.5, one of which states that the site period is equal to four times the shear wave travel-time through the material from the underlying rock to the surface. This site period approach recognizes that deep stiff/dense soils can exhibit long-period site response characteristics compared to shallower deposits. Unlike in most seismic codes which use only V_{s30} (the average shear wave velocity of the upper 30 m soil deposit) to define soil classes, NZS1170.5 considers shear wave velocity and the natural period of the soil as detailed in Table 1.

Table 1. Current soil class classification in NZS1170.5

Soil Class	Description	Shear Wave Velocity, V_s (m/s)	Period, T_{low}[1](s)
A	Strong rock	$V_{s30} > 1500^2$	--
B	Rock	$V_{s30} > 360$	--
C	Shallow soil	--	< 0.6
D	Deep/soft soil	--	> 0.6
E	Very soft soil	$V_{s10} > 150^3$	--

[1] the low amplitude natural period.
[2] the average shear wave velocity for the upper 30 m.
[3] the average shear wave velocity for the upper 10 m.

As shown in Table 1, two quantitative parameters (i.e. V_s or T_{low}) are utilized to determine the soil class. Equation (3) is used to calculate the related shear wave velocity V_s for class C and D soil, where H is the depth of the soil deposit in metres. Meanwhile, to be consistent, V_{s30} of each soil class will be used in this study.

$$T = \frac{4H}{V_s} \qquad (3)$$

Numerical model

DEEPSOIL, a one-dimensional site response analysis program [Hashash *et al.* 2011], is used to conduct the numerical analyses in order to investigate the effect of soil properties on the characteristics of ground motions transferred to the surface and the resulting demand on structures. In DEEPSOIL, a soil column can be broken up into individual layers, each of which is characterized using the corresponding soil properties (shear wave velocity). The bedrock motion is applied at the fixed base (assumed 30 m below the surface to be consistent with the code definitions), and the surface motion is recorded as the seismic waves travel vertically to the surface. In addition to the conventional frequency domain analysis (i.e. equivalent linear), non-linear time domain analysis is implemented in DEEPSOIL, which is equipped with sophisticated soil models (e.g. pressure dependent hyperbolic model [Hashash and Park 2001]).

A schematic representation of the soil model is shown in Figure 4. As shown in the figure, the soil between the surface and the bedrock is modelled in layers which are assigned appropriate material properties (such as shear wave velocity and density). DEEPSOIL also allows distinguishing between sand and clay [Darendelli 2001] for different soil layers. In order to capture the dynamic behaviour of soil deposits, non-linear time history analyses using the 'pressure dependent hyperbolic' model is conducted. As shown in Figure 4, the soil model assumes that the shear wave velocity, the unit weight, and material damping of the bedrock are 1500 m/s, 2.56 t/m^3, and 2%, respectively. Seismic motions are applied at the bedrock which travels vertically to the surface where the elastic acceleration response spectrum is generated.

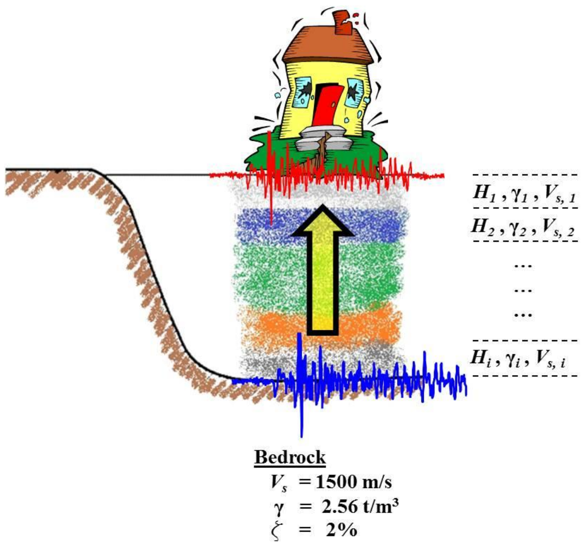

***Figure 4:** Schematic of soil model.*

Input bedrock motions

To accurately capture the record-to-record variations in seismic excitations, a range of ground motions are used. The SAC (Structural Association of California) suite [Somerville *et al.* 1997] used in this investigation includes a group of twenty ground motion records from the past earthquakes in California. These ground motions were recorded at different distances from the source faults of different characteristics, and include a significant range of possible motions with different frequency contents. In addition, these ground motions also inherit a reasonable variation in intensity.

Figure 5 presents the response spectra of the 20 SAC motions used in this study. As shown, the SAC set includes a range of typical moderate to strong earthquakes with average peak ground acceleration (PGA) equal to 0.22g. In an attempt to consider varying degrees of nonlinearity in the soil, the ground motions are scaled to different intensity levels, which will also allow conclusions to be drawn with regards to the effect of local seismicity. For example, a large portion of the analyses were conducted with the ground motions scaled to a PGA of 0.3g to represent a moderate seismicity region in New Zealand (i.e. Christchurch Z = 0.3). Similarly, the ground motions were also scaled to different levels between 0.1g and 0.5g to cover the range of seismicity within NZ.

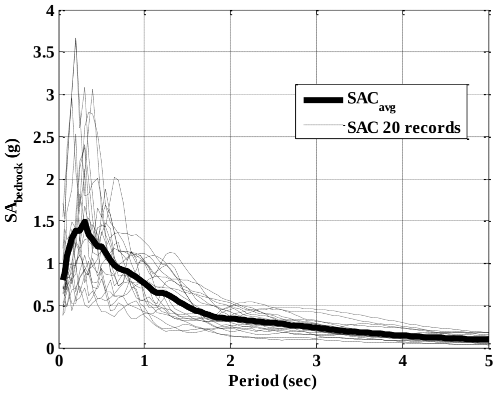

Figure 5: ***Response spectra of the 20 SAC motions (i.e. input bedrock motions).***

PARAMETRIC ANALYSES

The analysis process focused on the variation of a number of soil properties and input motion intensity which would provide insight into the sensitivity of the model. This would also generate a wide range of spectral shapes that would be representative of the actual variation in local geology.

Over 2,000 analyses in total were conducted in this study and the results shown below are an average of all 20 ground motions for the corresponding combination of the period under consideration, ground motion intensity and subsoil class. For example, in the determination of the normalised acceleration response spectra for soil class B given in Figure 6, five soil models with different average shear wave velocities falling within the definition of soil class B subject to all 20 SAC motions were performed. The response of soil class B reported herein (the bold solid line) is the average of the 5 soil models (representing the 100 analyses; i.e. 20 ground motions x 5 soil models).

Effect of soil properties

It is known that the dynamic response of soils can be greatly affected by confining pressure (i.e. K_o, horizontal earth pressure coefficient), the over consolidation ratio (OCR) and the level of clay plasticity (i.e. PI, plasticity index). In a number of models, these properties were varied to identify the dependence/sensitivity of these parameters on the spectral response. As shown in Figure 7, the response was found to not be significantly affected by a variation in these parameters.

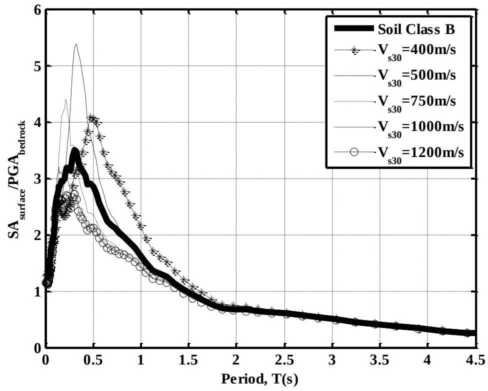

Figure 6: ***Normalised acceleration response spectra of 5 soil models with various shear wave velocity (the average response when subject to 20 SAC motions).***

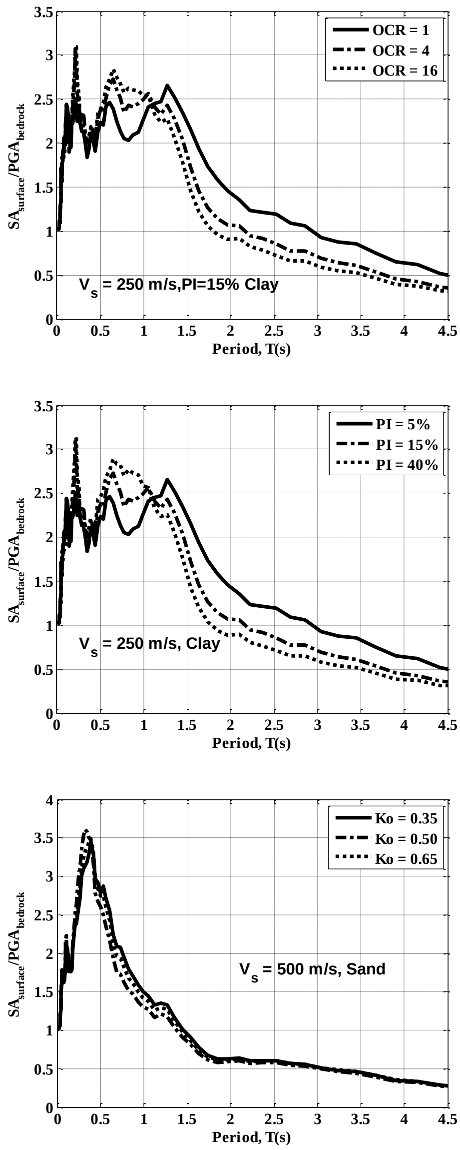

Figure 7: ***Normalised acceleration response spectra of various soil properties.***

Single layer vs. multi-layer soil deposits

In order to understand the difference between uniform and layered soil deposits, soil columns with single and multiple layers are arbitrarily developed for comparison such that their average shear velocities are equal. The average shear wave velocity of a multi-layered soil deposit is calculated using Equation (4) [ICC 2011].

$$V_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{V_{si}}} \tag{4}$$

where V_s = the average shear wave velocity

d_i = the thickness of soil layer i

V_{si} = the shear wave velocity of soil layer i

n = the number of soil layer

Soil deposits with two different average shear wave velocities for the upper 30 m (e.g. V_{s30_avg} = 350 m/s representing soil class C and 500 m/s representing soil class B) were considered for this investigation. For the single layer model, a 30m thick soil column with a constant shear wave velocity equal to the specified V_{s30_avg} was developed. On the other hand, the equivalent multi-layered model included a soil column composite of three 10m thick layers with varying shear wave velocities (i.e. 200, 500, and 800 m/s for V_{s30_avg} = 350 m/s; and 400, 500, and 600 m/s for V_{s30_avg} = 500 m/s). For other soil classes (i.e. A, D, and E), comparison with multi-layered modelling was not conducted because; (i) if layers of reasonably different properties are to be used, the shear wave velocity of at least one of the layers will have to be given an unrealistic (extremely high or low) value to ensure that the average shear wave velocity calculated by using Equation (4) is within the range of the soil class being considered; and (ii) if the layers are assigned properties very close to each other to ensure the average shear wave velocity falls in the narrow range of the soil class being considered, the multi-layered model is already close to a single layer model and a separate comparison is not deemed necessary.

Figure 8 below shows the comparison of the normalised acceleration response spectra between single and multi-layered soil deposits with different average shear wave velocities. As can be seen, except for the noticeable overestimation of spectral acceleration for the Class C soil (i.e. $V_{s30\ avg}$ = 350 m/s) between 0.5 s and 1 s, the single layer soil model captures the behaviour of the layered deposit quite well. Scrutiny of the individual results did not reveal any specific reason for the localised overestimation by the single layer model with 350 m/s shear wave velocity. Given that the predictions are close for the soil class B model and also at all other periods for the soil class C model as well, a single layer soil column with a constant shear wave velocity over the top 30m depth is thought to reasonably capture the behaviour of a multi-layered soil deposit of equal average shear wave velocity; and is hence used in the parametric analysis conducted hereafter.

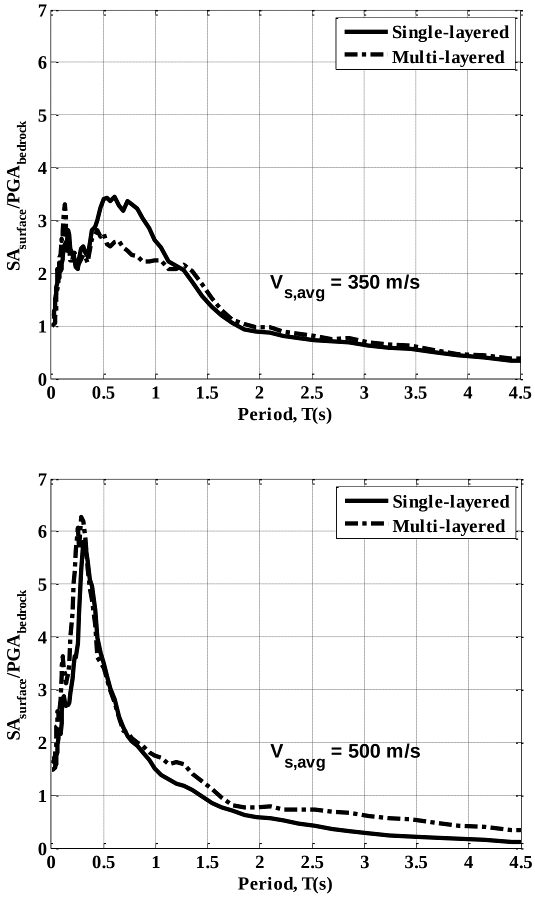

Figure 8: Normalised acceleration response spectra of single- and multi-layer soil deposits.

Effect of bedrock motion intensity

Past earthquakes such as those in Loma Prieta and Mexico City have been reported to show evidences of significant amplification of low PGA bedrock motions in soft clay sites. To investigate the change in the surface motion due to a change in the intensity of the input seismic excitation, PGA of motions applied at the bedrock level (classified by *Z*) was varied within a range to ensure that the soil deposits were forced to respond to different extents of nonlinearity.

As shown in Figure 9, reduction in the level of input bedrock motion intensity results in a greater amplification of the short period response spectrum for hard soil. On the other hand, the input bedrock motion's peak acceleration amplitude is not amplified by a soft soil deposit unless the input intensity in small enough; for larger input intensity the ground motion acceleration decreases while travelling through the soil deposit to the surface. Nevertheless, even in soft soil the ratio of the surface motion's spectral acceleration to the bedrock motion PGA increases with a reduction in input motion intensity; and this increase is particularly more prominent in class E soils in which the output to input ratio of low intensity ground motions is found to be consistently greater across all periods. This is because low intensity bedrock motion allows the soil to respond more in the linear range (i.e. the extent of nonlinear response is smaller); thereby reducing the levels of stiffness degradation in the soil column and consequently resulting in a greater surface acceleration to the bedrock acceleration ratio.

However, this does not corroborate the hierarchy adopted by the NZS1170.5. Despite the increased amplification of less intense ground motions, the maximum acceleration amplification observed in the soil class E (even for the lowest intensity Z = 0.1) is still much less than the smallest amplification observed in the soil class B. Hence, using larger acceleration for softer soil (as in NZS1170.5) cannot be justified on the basis of difference in the extent of nonlinearity in the soil response caused by ground motions of different intensity.

According to the above parametric study, the response of soil columns due to seismic excitation is significantly affected by two variables: the shear wave velocity of the soil, and the intensity of the input motions.

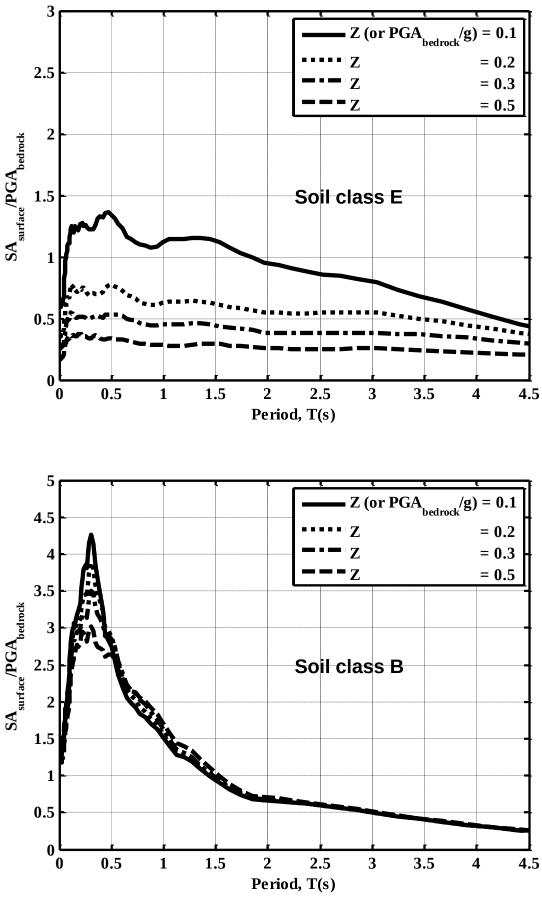

Figure 9: Effect of input motion intensity on soft soil class E and hard soil class B.

Therefore, a single layer soil column with 30m depth and constant shear wave velocity will be used to distinguish the different soil classes in the following analyses, which are aimed at investigating: 1) the amplification in peak acceleration from the bedrock to the soil surface; and 2) the structural response due to the altered frequency content of the surface motions. It is worth noting that the assumption of bedrock at 30 m below the surface is a limiting consideration in this study, which is adopted mainly for simplicity and to be consistent with the soil class definitions provided in design codes. It can be argued that deep soil deposits (much greater than 30 m) may exhibit higher mode effects with reduced period and therefore increased acceleration response; this needs further investigation to verify.

RESULTS AND DISCUSSION

An extensive numerical investigation is conducted using the soil model refined based on the results of the parametric study. The results and comparison with NZS1170.5 is presented in this section. Furthermore, a revised design procedure is also proposed, followed by a design example.

Comparison with the NZS1170.5 spectral shape factor

In order to evaluate the validity of the soil effect in NZS1170.5 (i.e. the spectral shape factor $C_h(T)$), normalised acceleration response spectra for different soil classes are generated using the scaled SAC motions (with an intensity of 0.3g which is the design *Z* factor for Christchurch). Figure 10 shows the average normalised spectral response from the models for a range of soil properties which fall in the corresponding soil classes as listed previously in Table 1. Note that the spectral acceleration in these curves are normalized with respect to the bedrock PGA (not the surface motion PGA) to provide a direct comparison with the $C_h(T)$ curves of NZS1170.5 (Figure 1); hence they do not converge to 1 at zero period.

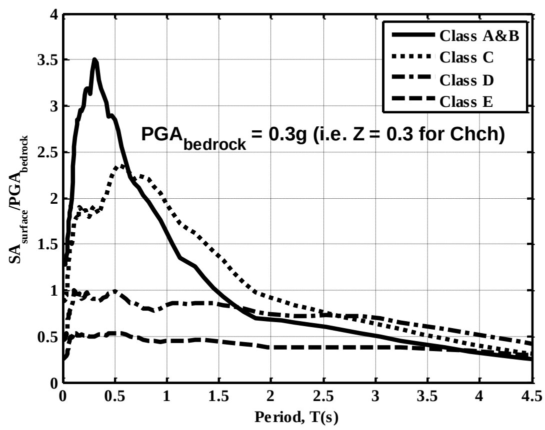

Figure 10: Spectral shape curves for different soil classes.

It can be seen that there are some fundamental differences (and some similarities in trend) when comparing Figure 10 to the NZS1170.5 spectral shape factor curves shown in Figure 1. The comparison indicates that:

- The hierarchy of the soil response in short period range is opposite to that given in NZS1170.5. In this case, stiffer soil deposits are found to amplify the spectral acceleration response significantly more than the soft soils do in the same period range.
- The results show that the soft soils amplify the long period response more than the hard soils. This trend is consistent with the current NZS1170.5 provisions, although the extent of difference seems to be exaggerated in NZS1170.5.
- Harder soils generate significantly greater response amplification than the soft soils. In particular, the acceleration demand of low period structures on soil classes A & B are significantly underestimated by NZS1170.5.
- Similarly, $C_h(T)$ curves of NZS1170.5 seem to overestimate the demands for short period structures on very soft soil (classes D and E).

In NZS1170.5 the spectral shape curves are typically employed for an equivalent static analysis method which is limited to regular structures that are less than 10 m in height. This indicates that the spectral curves are commonly applied to short period structures (a period below 1 s). Therefore, the relative magnitudes of short period spectral acceleration are very important. As the analysis results have shown significant variations in the response amplification in this period range, it is subjected to greater scrutiny in this study.

To further evaluate the discrepancy between the observed results and NZS1170.5 and to ascertain if NZS1170.5 provisions are likely to lead to any unsafe design, structural demands for different structural periods and input intensities are obtained from the analysis and calculated using the

NZS1170.5 provisions. The differences between these two values are tabulated in Table 2. Zone factors (Z) of 0.1, 0.3, and 0.5 which approximately correspond respectively to the Auckland, Christchurch, and Wellington regions, were investigated. The table shows conservative estimates by NZS1170.5 in light grey, with comparable estimates (within 50%) in grey and un-conservative estimates in dark grey. It can be seen that for soft soils (soil classes D and E), the current provisions result in safe design regardless of the intensity of the input motion. On the other hand, for hard soil (classes A and B) the current code provisions consistently underestimate the demand and result in an unsafe design regardless of the input intensity.

Dependency on intensity of input motion

The response of soil to seismic motions is severely affected by the amount of shear strain induced during seismic motions. High intensity motions induce large strains and therefore significant nonlinear behaviour. This in turn reduces stiffness and increases hysteretic damping, reducing the ability of the soil to transmit force to the surface and structure above. This has been outlined previously in the parametric study section where significant variation in the soil behaviour has been observed.

However, the intensity or location dependency is not implemented in current NZS1170.5 provisions where the same $C_h(T)$ curves are utilized no matter where the building is located (i.e. the Z factor). The transformation of the shaking intensity from the bedrock to the ground surface is schematically shown in Figure 11. It normalises the peak surface acceleration with the peak bedrock acceleration for a range of shear wave velocities to show the relative amplification or de-amplification of the shaking intensity.

Table 2. Difference between the NZS1170.5 structural seismic demands and the predicted results

Z	Period (s)	Soil Class A & B	Soil Class C	Soil Class D	Soil Class E
0.1	0.1	-10%	19%	41%	61%
	0.2	-42%	-2%	30%	58%
	0.3	-81%	-21%	22%	59%
	0.5	-71%	-107%	38%	55%
	1	-61%	-107%	-12%	63%
	2	-26%	-22%	-16%	42%
0.3	0.1	0%	45%	70%	83%
	0.2	-27%	37%	69%	83%
	0.3	-49%	35%	70%	83%
	0.5	-78%	-16%	67%	82%
	1	-73%	-63%	56%	85%
	2	-28%	-41%	31%	77%
0.5	0.1	10%	60%	79%	89%
	0.2	-19%	52%	80%	88%
	0.3	-29%	56%	80%	89%
	0.5	-65%	22%	79%	89%
	1	-80%	-20%	72%	91%
	2	-33%	-28%	55%	84%

conservative estimates by NZS1170.5,
comparable estimates (less than 50%),
un-conservative estimates (greater than or equal to 50%).

Figure 11 illustrates how an increase in input intensity (Z= 0.5, for example) results in significant de-amplification of the soil response; particularly for soft soils. It is expected as high intensity motions induce large shear strains in softer soils which takes the response into nonlinear regime. This causes significant degradation of the soil stiffness and increase in damping. Conversely, lower input motion intensity allows the softer soils to behave more linearly (small amounts of stiffness degradation). This can be seen by amplification of the input motion of up to 150% for soil class C.

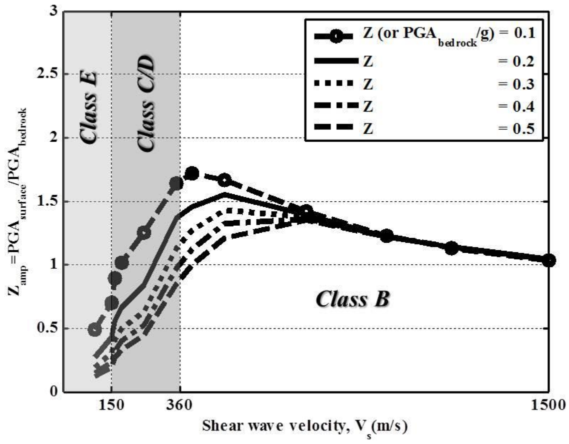

Figure 11: Relationship between normalised PGA and shear wave velocity for various Z factors.

At high levels of shear wave velocity, Figure 11 shows all curves (corresponding to different Z factors) asymptotically converge to the value of unity. This behaviour is expected as an infinitely stiff rock layer should be simply transferring the applied bedrock motion without amplification or de-amplification. On the other hand, there is a clear tendency of de-amplification when shear wave velocity is very small. This is similar to the basic concept of base isolation. All curves are heading towards the origin; which is in line with the dynamic principle that the acceleration response of an infinitely flexible system (represented by shear resistance V_s = 0) is zero.

PROPOSED REVISION TO THE CURRENT METHOD OF CONSIDERING LOCAL SITE EFFECT

The results discussed in the previous section indicate that the seismic demand of a structure depends significantly on the soil class and the intensity of input motions, which are currently represented via the $C_h(T)$ and Z factors in NZS1170.5. More importantly, the response of soil deposits to bedrock motion was also found to be significantly affected by the intensity level of the input bedrock motion. However, this intensity dependency is not currently included in the NZS1170.5, where the response spectral shape factor $C_h(T)$ is highly simplified as the same shape is used for all over the country, irrespective of the local seismicity, i.e. Z factors.

Introduction of an intensity amplification factor Z_{amp}

To capture the above-explained intensity effect, an intensity amplification/de-amplification factor is required in addition to the spectral shape factor. The authors advocate for such a factor (Z_{amp}) to be used in the next revision of the NZS1170.5 in order to account for the change in the PGA of the transferred motion. It was also realised during the development of NZS1170.5 that the spectra for soft soil classes (D and E) were intensity dependent. This aspect however was not included in the 2004 revision reportedly to avoid an increase in complexity of the method [McVerry, 2003].

The proposed method in this study identifies the need for a simplistic strategy. With an attempt to achieve this while also accurately capturing the soil response more comprehensively, the proposed revised method only makes a minor change to the equation to that currently outlined in NZS1170.5. The proposed expression for the elastic site hazard spectrum for horizontal loading is:

$$C(T)=C'_h(T)*Z*Z_{amp}(Z, soil)*R*N(T, D) \quad (5)$$

As is obvious from Equation (5), the hazard factor Z, return period factor R, and the near fault factor $N(T,D)$ used currently in NZS1170.5 are retained. However, the spectral shape factor $C_h'(T)$ is revised (detailed in the next section) and a new PGA amplification factor (i.e. Z_{amp}) is introduced. As seen in Figure 11, the value of Z_{amp} depends on the soil stiffness (i.e. shear wave velocity) and the input motion intensity. Interpolated values of Z_{amp} for different soil classes and Z values are provided in Table 3.

Table 3. Interpolated median and the 84th percentile (in bracket) values of Z_{amp} for different soil classes and Z values

Z	Soil Class			
	A & B	C	D	E
0.1	1.45 (1.62)	1.45 (1.60)	0.96 (1.03)	0.60 (0.67)
0.2	1.36 (1.50)	1.10 (1.21)	0.61 (0.69)	0.35 (0.40)
0.3	1.29 (1.43)	0.89 (0.98)	0.46 (0.52)	0.25 (0.29)
0.4	1.23 (1.37)	0.75 (0.84)	0.36 (0.42)	0.19 (0.24)
0.5	1.19 (1.30)	0.65 (0.71)	0.30 (0.35)	0.16 (0.21)

Modified spectral shape factor $C_h'(T)$

As mentioned earlier, in NZS1170.5 the spectral shape factor $C_h(T)$ is independent of the seismicity of the region. To reflect the de-amplification tendency observed in the results of this study, while maintaining the concept of shape factor, a revised spectral shape factor $C_h'(T)$ is proposed, whose values can be obtained from Figure 12. It is worth noting that these curves represent the average results of a large number of analyses with different seismic intensity (i.e. Z) and different soil properties (such as OCR, earth pressure, and plasticity). Furthermore, the revised spectra are normalised using the intensity of surface motions (i.e. $PGA_{surface}$), which was different from that shown in Figure 10 (normalised using the intensity of bedrock motions, i.e. $PGA_{bedrock}$ or Z).

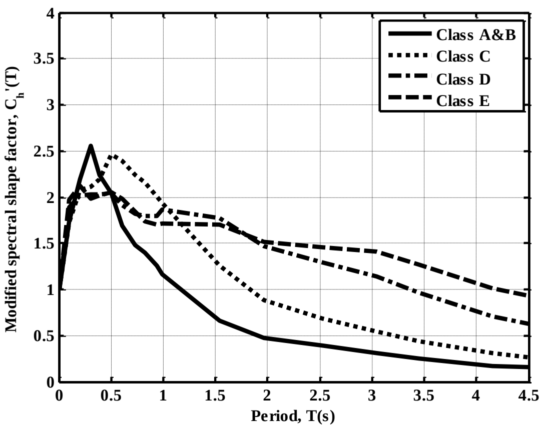

Figure 12: Revised spectral shape factor $C_h'(T)$.

Table 4 shows a tabulated form of the revised $C_h'(T)$ factor, which is obtained from Figure 12. Instead of the $C_h(T)$ table currently provided in NZS1170.5, the values of $C_h'(T)$ used in the calculation of seismic demand in the proposed method are to be taken from this Table.

Table 4. Proposed median and the 84th percentile (in brackets) spectral shape factors tabulated from the spectral shape curves (Figure 12).

Period	Soil Class			
(s)	A & B	C	D	E
0	1	1	1	1
0.1	1.73 (2.30)	1.72 (2.26)	1.88 (2.44)	1.97 (2.47)
0.2	2.19 (3.01)	2.08 (2.72)	2.01 (2.49)	2.12 (2.70)
0.3	2.56 (3.50)	2.11 (2.73)	2.03 (2.65)	1.99 (2.67)
0.4	2.23 (3.16)	2.20 (3.06)	2.03 (2.76)	2.01 (2.69)
0.5	2.04 (2.83)	2.47 (3.29)	2.04 (2.80)	2.05 (2.83)
0.6	1.69 (2.53)	2.39 (3.25)	1.91 (2.67)	1.98 (2.75)
0.7	1.48 (2.34)	2.24 (3.13)	1.82 (2.48)	1.84 (2.54)
0.8	1.40 (2.14)	2.16 (3.03)	1.79 (2.35)	1.74 (2.32)
0.9	1.25 (1.99)	2.00 (2.91)	1.79 (2.37)	1.70 (2.27)
1	1.17 (1.88)	1.93 (2.86)	1.87 (2.51)	1.71 (2.36)
1.5	0.66 (1.13)	1.26 (2.35)	1.77 (2.68)	1.70 (2.46)
2	0.48 (0.85)	0.88 (1.75)	1.46 (2.24)	1.52 (2.13)
2.5	0.39 (0.78)	0.69 (1.54)	1.30 (2.27)	1.46 (2.13)
3	0.31 (0.63)	0.55 (1.28)	1.14 (2.25)	1.41 (2.25)
3.5	0.25 (0.54)	0.44 (1.07)	0.97 (2.05)	1.27 (2.17)
4	0.17 (0.38)	0.30 (0.76)	0.71 (1.64)	1.01 (1.93)
4.5	0.15 (0.34)	0.27 (0.67)	0.63 (1.48)	0.93 (1.83)

Comparison between the current and proposed methods

The difference between the proposed method and the current NZS1170.5 method can be illustrated through a design example in which the elastic site hazard spectra (i.e. $C(T)$) is calculated using these two methods. Table 5 demonstrates thus calculated $C(T)$ values for a typical two storey building with an approximate period of 0.4 s, and located in Christchurch on two different soil types. The return period factor (R) and near-fault factor (N) are both assumed to be unity.

Table 5. Design examples for Christchurch

	Class B soil		Class E soil	
	Revised	NZS	Revised	NZS
Z	0.3	0.3	0.3	0.3
$C_h(T)$	2.23	1.89	2.01	3.0
Z_{amp}	1.29	--	0.25	--
$C(T)$	0.86	0.57	0.15	0.90
Difference	-51%		83%	

Table 5 illustrates the trends that have been observed as part of this study. It is seen that the current consideration underestimates the seismic demand on hard soil (i.e. class B) by 51%. It is worth noting that the above proposed $C_h'(T)$ factors are determined using the median values, the difference will be substantially greater (-139%) when the upper characteristic (84th percentile) value is taken into account. This is a very un-conservative estimate which may lead to unsafe design. On the other hand, the demand on soft soils (i.e. class E) is overestimated by 83%, potentially leading to a structure which is safer than it is required to be (provided the soft soil does not liquefy).

Interestingly, this is in line with the damage observed in Christchurch in the recent earthquake series. Although the ground motions induced in the February earthquake were more intense than what the structures were designed for (consequently the structures were expectedly damaged to different extents), the nature of damage observed in the suburbs located on soft soil and hard rock was qualitatively consistent with the findings of this study. Buildings in the city and eastern suburb where the soil was soft were mainly subjected to ground failure; there was scarcely any evidence of severe damage to buildings without excessive deformation of the underlying soil. This indicates that these buildings were overdesigned, which rendered the strength of these buildings greater than intended; and more importantly much greater than the capacity of the soil. On the other hand, there were plenty of buildings in the rocky suburbs such as Port Hills, Mt Pleasant, etc. which suffered damage to the superstructure without any noticeable soil deformation. Interestingly, the damages (such as tiles falling from the roof, collapse of heavy boundary walls, severe damage to building contents) indicated that the acceleration demand on these buildings were substantially higher than they were designed to resist. Also in Port Hills, there were several examples of dislodged rocks and boulders rolling down hill to severely damage houses in Sumner, which indicates that the surface acceleration in the hilly suburb was large enough to overturn the rocks from their original stable positions.

CONCLUSIONS

The investigation has shown that the current approach adopted in NZS1170.5 to account for local site effects is unable to represent fully the variation in structural seismic demand for different soil classes. It has been identified that the design demand at the surface depends on the intensity of the bedrock motion in addition to the stiffness of the underlying soil.

The spectral shape curves currently provided in NZS1170.5 suggest a soil class hierarchy of increased amplification of bedrock motion by softer soils. This study has found that this is true only in the long-period range. It has also shown that hard rocky deposits (soil class A & B) produce large amplifications at short periods that are greater than the short-period response of soft soils. This indicates that the current seismic consideration is not conservative for stiffer soils and is over-conservative for soft soils. Such effects might be used to explain to some extent why low rise residential properties were severely damaged by intense ground motions on stiff soils in the Port Hills region during the Christchurch earthquakes.

A revised approach to account for the local site effects in seismic design has been proposed which overcomes the limitations of the current design provisions. The proposed method introduces a new ground motion amplification factor Z_{amp} and modifies the existing spectral shape factor, $C_h'(T)$. Through a couple of examples, the new method has been shown to more accurately capture the local soil response.

It is noted that the proposed method was developed based on an extensive numerical investigation taking into account the effect of different parameters on the site response. All (more than 2,000) analyses were conducted using advanced state-of-art soil models available in the nonlinear site response analysis program DEEPSOIL. The analytical predictions were found to capture reasonably the trends shown by the real ground motions recorded in different soil classes. Hence, the outcome of this investigation could be relied upon to represent the local soil effects more accurately than the existing provisions. Nevertheless, further verification using different analytical tools and experimental data may instil more confidence on the outcome.

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