Effect of Soil Type on Seismic Demand

A.K. Loye, S.J. Evans, S.L. Lin & R.P. Dhakal
University of Canterbury, Christchurch

2013 NZSEE
Conference

ABSTRACT: This paper investigates the validity of the soil considerations used in the determination of seismic demand as part of NZS1170.5, which currently specifies seismic design spectra corresponding to 5 different soil types. According to the current provisions stipulated in NZS1170.5, for all natural periods, the building demand for soft soil is either equal to or greater than that for hard soil. It is noted that 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 1-D nonlinear site response analysis 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 column (represented by its shear wave velocity, V_s) are the main parameters affecting the surface response. 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 soil types 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 seismic structural demand is proposed and demonstrated.

1 INTRODUCTION

In most seismic prone countries, the minimum seismic design requirements and different methods to determine design actions in building components (e.g. equivalent static design, time-history analysis, and non-linear static analysis) specified in the codes are enforced legally. Among them, the equivalent static design procedure, where a lateral force is calculated and then applied to the structure as a set of static equivalent force, is widely accepted in most seismic codes. In this approach, the lateral design seismic force is calculated as a product of the building weight and a coefficient, which depends on the building period, use (i.e. importance factor), inherent ductility, local seismicity and soil condition.

In New Zealand, the coefficient is termed as the elastic site hazard spectrum the determination of which is detailed in the Australia/New Zealand Standard: Structural design actions, Part 5: Earthquake actions - New Zealand (NZS 1170.5, (SNZ 2004). For horizontal loading, this coefficient $C(T)$ is evaluated as shown in Equation (1) below,

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

where $C_h(T)$ = the spectral shape factor, Z = the zone factor, R = the return period factor, and N = the near-fault factor. Among them, the local site effect is implemented via the spectral shape factor $C_h(T)$ that characterises the seismic response for five different soil classes (alphabetically categorised from A/B: rock to E: soft soil) as shown in Figure 1. The $C_h(T)$ factors are the response spectrum values $S_a(T)$ from hazard analyses normalised by Z (McVerry 2003), where Z is defined as half of the spectral acceleration at 0.5sec (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. hazard level, or seismic intensity) in current NZS1170.5. In other words, the $C_h(T)$ curve (Figure 1) utilized in Auckland (low seismic zone) is exactly the same as that in Wellington (high seismic zone).

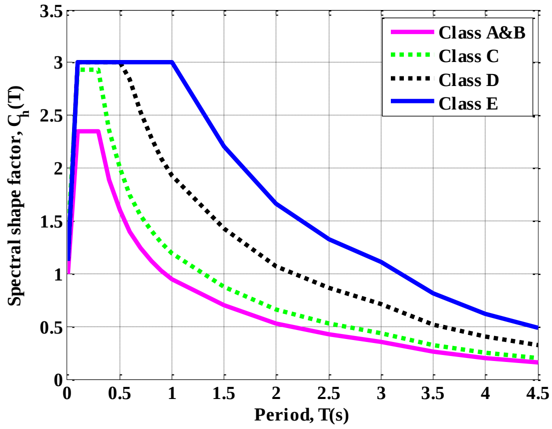

Figure 1: Spectral shape factor in NZS1170.5

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" – has been extensively observed in previous earthquakes (Loma Prieta 1989 (Seed et al. 1990) and Mexico City 1985 (Dobry and Vucetic 1987), for example) and therefore is acknowledged in most seismic codes. 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 deamplified at the ground surface.

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

The tendency of soft soil to amplify earthquake motions on the bedrock beneath, which is commonly employed in most seismic codes (NZS1170.5, for example) can be tracked to some evidences observed in the previous earthquakes, such as Loma Prieta and Mexico City earthquakes, as mentioned earlier. Nevertheless, there are other evidences which show 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. Most recently, the acceleration response spectra 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.

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

2 METHODOLOGY

To quantitatively capture the structural demand due to different soil types, sets of numerical analyses with varying soil properties and input motions are conducted.

Five site subsoil classes are defined in NZS 1170.5. 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 site period is equal to four times the shear wave traveltime through the material from the surface to underlying rock. 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 30m soil deposit) to define soil classes, New Zealand approach considers shear wave velocity and soil period.

2.1 Numerical Model

DEEPSOIL, a 1-D 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 codal definitions), and the surface motion is recorded as the seismic waves travelling 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 more sophisticated soil models (e.g. pressure dependent hyperbolic model (Hashash and Park 2001).

In this study, 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), which is selected for soil layers above the bedrock. In order to capture the dynamic behaviour of soil deposits, non-linear time history analyses using ‘pressure dependent hyperbolic’ model is conducted. Meanwhile, 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.

2.2 Seismic Input Motions

To accurately capture the effect of soils subject to seismic excitations, a range of ground motions are used. The SAC (Structural Association of California) suite (Somerville et al. 1997) 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 allow a significant range of possible motions with different frequency contents to be considered. In addition, these ground motions also inherit a reasonable variation in intensity.

The SAC motions used in this study represent a range of typical moderate to strong earthquakes (with average peak ground acceleration (PGA) equal to 0.22g). Motions have been intensity scaled to allow comparison between relative intensity motions, with an attempt to consider varying degrees of nonlinearity in the soil. For example, a large portion of the analyses were scaled to a PGA of 0.3g to represent a moderate seismicity region in New Zealand (i.e. Christchurch Z=0.3).

3 PARAMETRIC STUDY

The analysis process focused on the variation of a number of soil properties, deposit depth, 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 soil geology that exists.

Over 2000 analyses in total were conducted in this study and the results shown below were an average of response for all 20 ground motions. For example, in the determination of the normalised acceleration response spectra for soil class B, five soil models with different average shear wave velocities subject to all 20 SAC motions (i.e. 100 analyses in total) were performed. Response of soil class B was then the average.

3.1 Effect of Soil Properties and Number of Layers

It’s 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. The response was found to not be significantly affected by a variation in these parameters.

In order to understand the effect of number of soil layers, soil deposit with single or multiple layers are arbitrarily developed such that their average shear velocities are equal. Soil deposits with two different average shear wave velocities for the upper 30m (e.g. V_{s30_avg} = 350 and 500 m/s) 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 (e.g. 200, 500, and 800 m/s for V_{s30_avg} = 350 m/s; 400, 500, and 600 m/s for V_{s30_avg} = 500 m/s). The comparison between the response spectra of the single- and multi-layered soil deposits showed some random minor difference, but no consistent trend was found to enable the difference to be quantified. Hence, a single-layered soil column with constant shear wave velocity over the 30m depth is assumed to effectively capture the behaviour of multi-layered soil deposit of equal average shear wave velocity.

3.2 **Input Motion Intensity Effect**

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. However, there are other evidences which show the opposite trend, as discussed previously. To investigate the change in the predicted response due to different intensity of the seismic excitation, PGA of motions at the bedrock level (representative of the *Z* factor) was varied within a range to ensure that the soil deposits were forced to respond to different extents of nonlinearity.

As shown in Figure 2, reduction in the level of input intensity results in a greater amplification of the spectral acceleration in the short period region of the response spectrum; this change is particularly more prominent in class E soils. 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 amplification (i.e. surface acceleration to the bedrock acceleration ratio). When comparing to hard soils, it is seen that the acceleration amplification in the softer soil is still equivalent to or less than the amplification in the hard soils.

According to the above parametric study, it has been identified that 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.

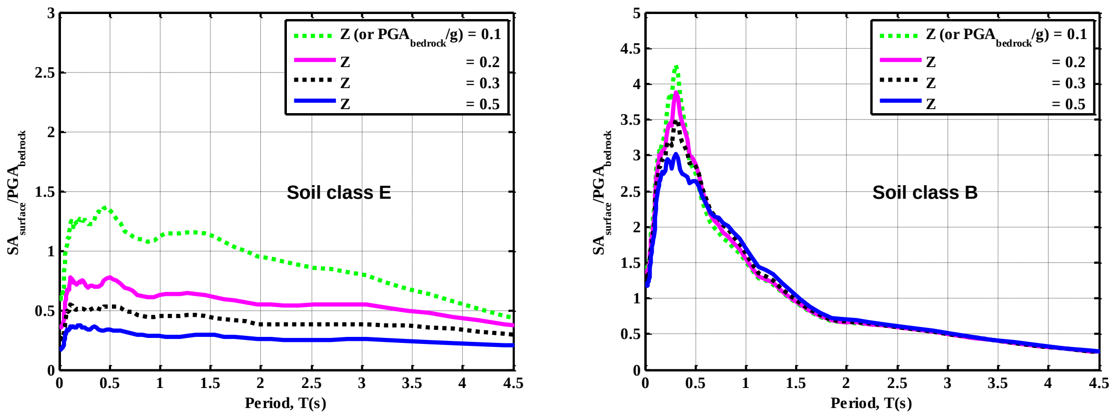

Figure 2: Effect of input motion intensity on soft soil class E and hard soil class B

Therefore, a single layer soil column with 30m depth and constant shear wave velocity is used to distinguish the different soil classes in the following analyses to further investigate two distinct components: 1) the amplification in peak acceleration from the bedrock to the soil surface, 2) and the structural response due to the altered frequency content of the surface motions. It is worth noting that the assumption of bedrock at 30m 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 the design codes. It can be argued that deep soil deposits (much greater than 30m) may exhibit higher mode effects with reduced period and therefore increased acceleration response; this needs further investigation to verify.

4 RESULTS AND DISCUSSION

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

4.1 Comparison with $C_h(T)$ in NZS1170.5

In order to evaluate the validity of the soil effect in NZS1170.5 (i.e. the $C_h(T)$ factor), normalised acceleration response spectra for different soil classes were generated using the scaled SAC motions (with an intensity of 0.3g which is the design Z factor for Christchurch). Figure 3 shows the average normalised spectral response from the models for a range of soil properties which fall in corresponding soil classes. 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 in NZS1170.5 (Figure 1); hence they do not converge to 1 at zero period.

It can be seen that there are some fundamental differences (and some similarities in trend) when comparing Figure 3 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 types A & B are significantly underestimated by NZS1170.5.
- Similarly, NZS1170.5 $C_h(T)$ curves seem to overestimate the demands for short period structures on very soft soil (type D and E).

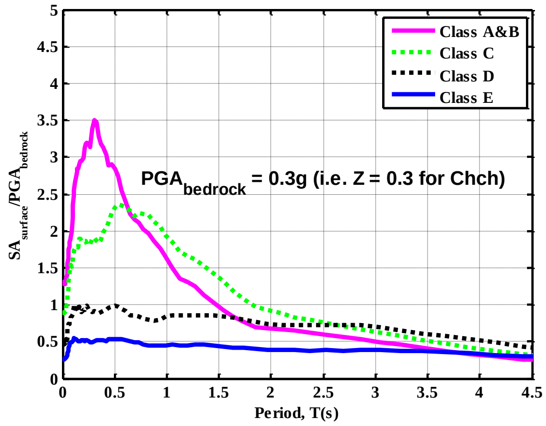

Figure 3: Spectral shape curves for different soil classes

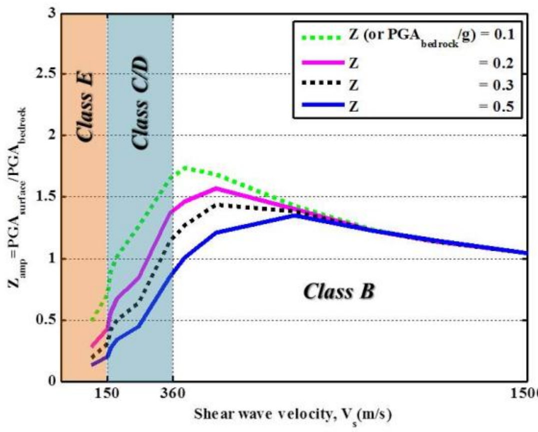

Figure 4: Relationship between normalised PGA and shear wave velocity for various Z factors

To further support the observations listed above, investigations using various soil models (in particular for soil class C and D) and input motions (motions on bedrock, for example) are underway. Preliminary results show similar trends, thereby further endorsing these observations. More detailed results and discussion will be published once available.

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 10m in height. This indicates that the spectral curves are commonly applied to short period structures (a period below 1s). 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.

4.2 Intensity Dependency

The response of soil to seismic motions is severely affected by the degree 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; thereby 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 4. 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.

Figure 4 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 increase nonlinear response. 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.

At high levels of shear wave velocity, Figure 4 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.

5 REVISED METHOD PROPOSAL AND DEMONSTRATION

The results discussed in the previous section indicate that the seismic demand of a structure depends significantly on the soil type and the intensity of input motions, which are currently represented in terms of the $C_h(T)$ and Z factor 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 included in the current NZS1170.5, where the response spectral shape factor $C_h(T)$ is highly simplified as the same shape and is used for all over the country, irrespective of the seismic region, i.e. Z factors.

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:

$$C(T)=C'_h(T)*Z*Z_{amp}(Z, soil)*R*N(T, D) \quad (2)$$

As is obvious from Equation (2), 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 (i.e. $C_h'(T)$ is revised (detailed in the next section) and a new PGA amplification factor (i.e. Z_{amp}) is introduced. As seen in Figure 4, the value of Z_{amp} depends on the soil stiffness (i.e. shear wave velocity) and the input motion intensity. The representative values of Z_{amp} for different values of Z are listed in Table 1.

Table 1. Interpolated median and the 84th percentile (in bracket) values of Z_{amp} for different soil type and Z values

Z	Soil Type			
	A & B	**C**	**D**	**E**
0.1	1.45 (1.62)	1.45 (1.60)	0.96 (1.03)	0.60 (0.67)
0.3	1.29 (1.43)	0.89 (0.98)	0.46 (0.52)	0.25 (0.29)
0.5	1.19 (1.30)	0.65 (0.71)	0.30 (0.35)	0.16 (0.21)

5.1 Revised Spectral Shape Factor $C_h'(T)$

As mentioned earlier, in the current NZS1170.5, the response spectral shape factor $C_h(T)$ is independent of the seismic region. To reflect the opposite amplification tendency as observed in the results of this study, but to maintain the concept of shape factor, a revised spectral shape factor $C_h'(T)$ is proposed, whose values can be obtained from Table 2, which is based on the curves shown in Figure 5. 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 3 (normalised using the intensity of bedrock motions, i.e. $PGA_{bedrock}$ or Z).

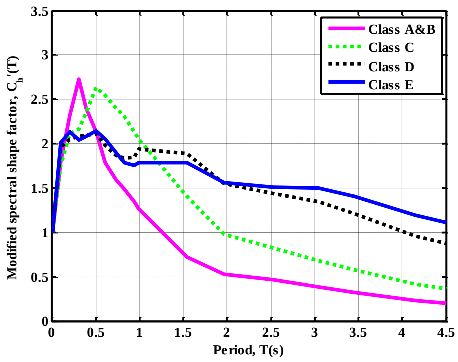

Figure 5: Revised spectral shape factor $C_h'(T)$

Table 2. Proposed median and the 84th percentile (in bracket) spectral shape factors tabulated from the spectral shape curves (Figure 5)

T *(s)*	Soil Type			
	A & B	**C**	**D**	**E**
0.1	1.73 (2.30)	1.72 (2.26)	1.88 (2.44)	1.97 (2.47)
0.3	2.56 (3.50)	2.11 (2.73)	2.03 (2.65)	1.99 (2.67)
0.5	2.04 (2.83)	2.47 (3.29)	2.04 (2.80)	2.05 (2.83)
0.7	1.48 (2.34)	2.24 (3.13)	1.82 (2.48)	1.84 (2.54)
0.9	1.25 (1.99)	2.00 (2.91)	1.79 (2.37)	1.70 (2.27)
1.0	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	0.48 (0.85)	0.88 (1.75)	1.46 (2.24)	1.52 (2.13)

5.2 Design Example

The difference between the proposed method and the current NZS1170.5 method can be illustrated through a design example in which the seismic weight coefficient (i.e. $C(T)$) is calculated using these two methods. Table 3 demonstrates thus calculated $C(T)$ values for a typical two storey building with an approximate period of 0.4s, and located in Christchurch on two different soil types. Both the return period factor (R) and near-fault factor (N) are assumed to be unity.

Table 3. Design example for Christchurch (T=0.4s)

	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 3 illustrates the trends that have been observed as part of this study. It is seen that the current consideration underestimates the seismic demand on the 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 results, the difference therefore will be more considerable when the variability is taken into account. This is a very un-conservative estimate which may lead to an 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 intended 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 February earthquake were more intense than what the structures were designed for (consequently the structures were expectedly damaged to different extents), the trend 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 also greater than that 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 response on these buildings were substantially higher.

6 CONCLUSION

The investigation has shown that the current consideration for local site soil effects in NZS1170.5 is unable to represent fully the variation in structural seismic demand for different soil types. It has been identified that the design spectra is influenced by the level of ground motion intensity in addition to the stiffness of the underlying soil.

The current spectral shape curves form a soil class hierarchy of increased amplification as the stiffness of the soil decreases. This study found that this is true only for softer soils in the long-period range. It also showed that hard rocky deposits (soil class A & B) produce large short period amplifications that are greater than the short-period response of soft soils. This indicates that the current seismic consideration is not conservative for stiffer rock type soils and is over-conservative for soft soils at high input intensity. Such effects might be used to explain in some ways that low rise residential properties were severely damaged by intense ground motions on stiff soils in the Port Hills during the Christchurch earthquakes.

Based on the above findings along with the limitations of the current soil consideration, a new approach has been proposed. By introducing the Z_{amp} factor and considering the soil effect on the spectral shape factor, $C_h'(T)$, the proposed method is able to more accurately capture local soil response.

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