SEISMIC ACTIONS ON ACCELERATION SENSITIVE NON-STRUCTURAL COMPONENTS IN DUCTILE FRAMES

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SUMMARY

Earthquake loadings standard NZS 1170.5:2004 has introduced new provisions for the design of building parts and non-structural components. The provisions include factors to define peak floor acceleration up the height of a building, and acceleration response amplifications for components that are quite different from overseas counterparts. In this study, acceleration demands on non-structural components located in ductile frame buildings are analysed under earthquake records from crustal and slab events, for design levels representing ultimate limit state and serviceability limit state.

A floor response spectra approach is used to study the demands on non-structural components. It is noted that the peak floor acceleration demands with respect to that of the ground are not amplified up the height of the building to the extent suggested in NZS provisions. The floor response spectra show peaks near the modal periods of the building indicating higher demands on the components with periods closer to the building period. However, NZS provisions fail to include this effect, since the spectral response amplification is defined independent of building period. Spectral demands exceed the NZS provisions at the fundamental periods of the buildings, more significantly at serviceability conditions, indicating potential failure of non-structural components with periods close to the building periods.

Following the analytical observations from the buildings considered in this study it is clear that the design provisions for non-structural components should be linked to the structural response for specific performance levels rather than the 'life-safety' performance level only that is currently adopted in the New Zealand design standard.

INTRODUCTION

The response of non-structural components (NSC) during earthquakes is an important consideration since failure of such components can result in more than half of the total damage cost [1]. As part of a performance-based earthquake engineering (PBEE) approach, the current design philosophy suggests a 'life-safety' performance objective to be achieved under severe earthquakes and accepts damage to the structure and non-structural components (except those posing life-safety hazards). However, under less intense shaking, with a performance objective corresponding to 'operational continuity', the structure is expected to suffer at most minimal damage. To maintain the building in an operational state it is crucial to avoid damage to NSC that could cause interruption to the functionality of the building. This was evident in the Nisqually 2001 earthquake, where damage to the non-structural components was reported to be extensive compared to the observed damage to structural components of buildings, reducing their overall performance [2].

During the past four decades, several rational methods have been developed to assess the demands on NSC, and state-of-the-art reports have been presented [3, 4]. Among these publications the 'Floor Response Spectrum' approach is well recognised. This approach involves time history analyses of the supporting structure to obtain an acceleration response history which is processed to provide spectra at the floor to which the component is attached. Even though the method is acknowledged to be complex and time consuming, it gives accurate results when: (i) the NSC considered is light enough (possibly with a mass ratio as low as 0.001[5]) to avoid any possible interaction with structural response; (ii) the behaviour of NSC is linear; and (iii) there are no multiple attachment points.

The response of non-structural components is affected by the dynamic response of the supporting structure to the input ground motions. When the structural response of primary and secondary systems is elastic, the floor response is generally amplified up the height of the building and the extent of amplification can be obtained using linear methods of analyses. However, several studies [e.g. 6, 7] have indicated that nonlinear behaviour of supporting structures significantly affects the demands on non-structural components and hence the use of linear methods in the analyses may lead to unrealistic designs. Recent studies [8, 9] included parametric investigation to understand the influence of various factors on seismic demands. The main parameters varied in these studies included the properties of the supporting structure; location of NSC within the structure; and damping of NSC. Their findings confirmed: (i) the amplification of floor acceleration up the height of the building under linear response; and (ii) approximately constant floor acceleration up the height of a building for non-linear structural response. Some studies have analysed interaction effects [10] and the response of linear and nonlinear NSC [8].

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General observations on acceleration demands due to the nonlinearity in the structure are: (1) nonlinearity of the structure tends to predominantly reduce the floor acceleration corresponding to the fundamental mode of vibration, and (2) amplification in peak component acceleration is likely when: (i) the component is on the lower stories of the building and localised nonlinear behaviour is observed [11], and (ii) the component period is close to that of one of the higher modes of the building [12, 13]. The contribution from higher modes was identified to be higher when the intensity of earthquake shaking was such that the structure responded elastically, or was just entering the inelastic range [12]. The amplification can possibly occur at floors associated with mode shape maxima.

From the above discussion, it is apparent that the floor response is amplified when the structure responds linearly but is amplified to a lesser extent when the structure responds non-linearly. Hence, design requirements for non-structural components should rather be linked to the response of the structural system under various ground shaking intensity levels.

The main objective of this paper is to evaluate acceleration demands of linear, light non-structural components in ductile frame buildings. The responses are studied under two intensity levels of ground shaking representing life safety or ultimate limit state (ULS) and operational or serviceability limit state (SLS). The numerical results are compared with the design provisions of NZS 1170.5:2004 [14]. Also, differences in floor responses due to differences in earthquake source type are studied.

CODE PROVISIONS ON NON-STRUCTURAL COMPONENTS DEMAND

Current code provisions recommend methods for obtaining equivalent static forces for the design of parts and components. The code provisions typically include two factors: (i) a factor to represent the variation of peak floor acceleration (PFA) up the height of the building; and (ii) a factor to represent the amplification of acceleration response of a component with a given period. In NZS 1170.5: 2004, the first factor is referred to as the floor height coefficient (FHC) and the second is referred to as the component amplification factor (CAF). The FHC is normalised with respect to the site hazard coefficient at the zero period ordinate of the response spectrum for building, C(0). Design actions are derived by modifying the response of the system by applying: (i) a 'response reduction factor' to account for the ductility of connections; and (ii) a risk factor to account for the consequences of failure. The NZS provisions suggest the following expression as the elastic demand on a component with a natural period T_p (Equation 1):

$$C_p\left(T_p\right)=C(0)C_{Hi}\,C_i\left(T_p\right) \tag{1}$$

where

C(0) = site hazard coefficient at zero period

C_{Hi} = floor height coefficient as in Figure 1

C_i (T_p) = component amplification factor as in Figure 2(a)

Equation 1 implicitly suggests the same floor response spectral shape up the building height irrespective of variations in acceleration spectral response up the building height.

Figure 1 shows the envelopes of floor height coefficient (FHC) as defined by NZS[14] and IBC [15]. NZS requires different envelopes for low and high rise buildings and they have two segments, a linearly increasing segment and a constant segment. This pattern is adopted to reflect higher mode effects. The distribution depends on the height of the attachment of the component relative to the height of the building (h/H). However, other source, such as IBC 2003[15], and SEI/ASCE-02 provisions based on NEHRP studies [16], defines a linear variation of peak floor acceleration up the height of the building. These provisions are partly based on analytical studies [6, 17], which indicated an increase in the floor acceleration in upper floors of the building; and partly based on data recorded in instrumented buildings [18, 19]. It can be appreciated that the NZS provision is highly conservative, particularly for high rise buildings, compared to IBC provisions with regard to floor height coefficient.

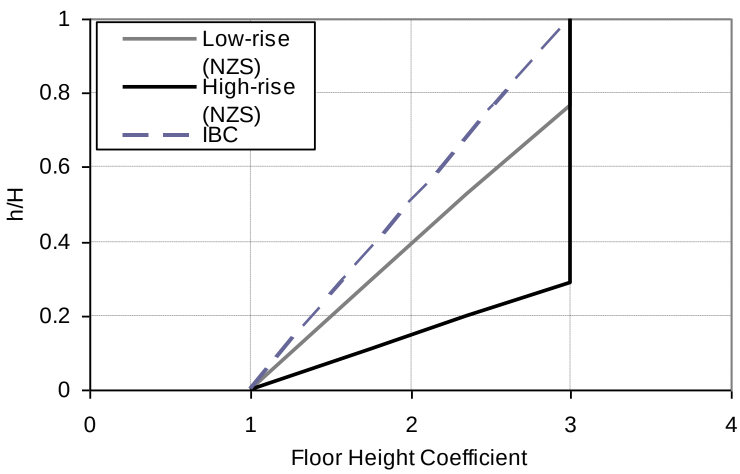


Figure 1: Variation of floor height coefficient.

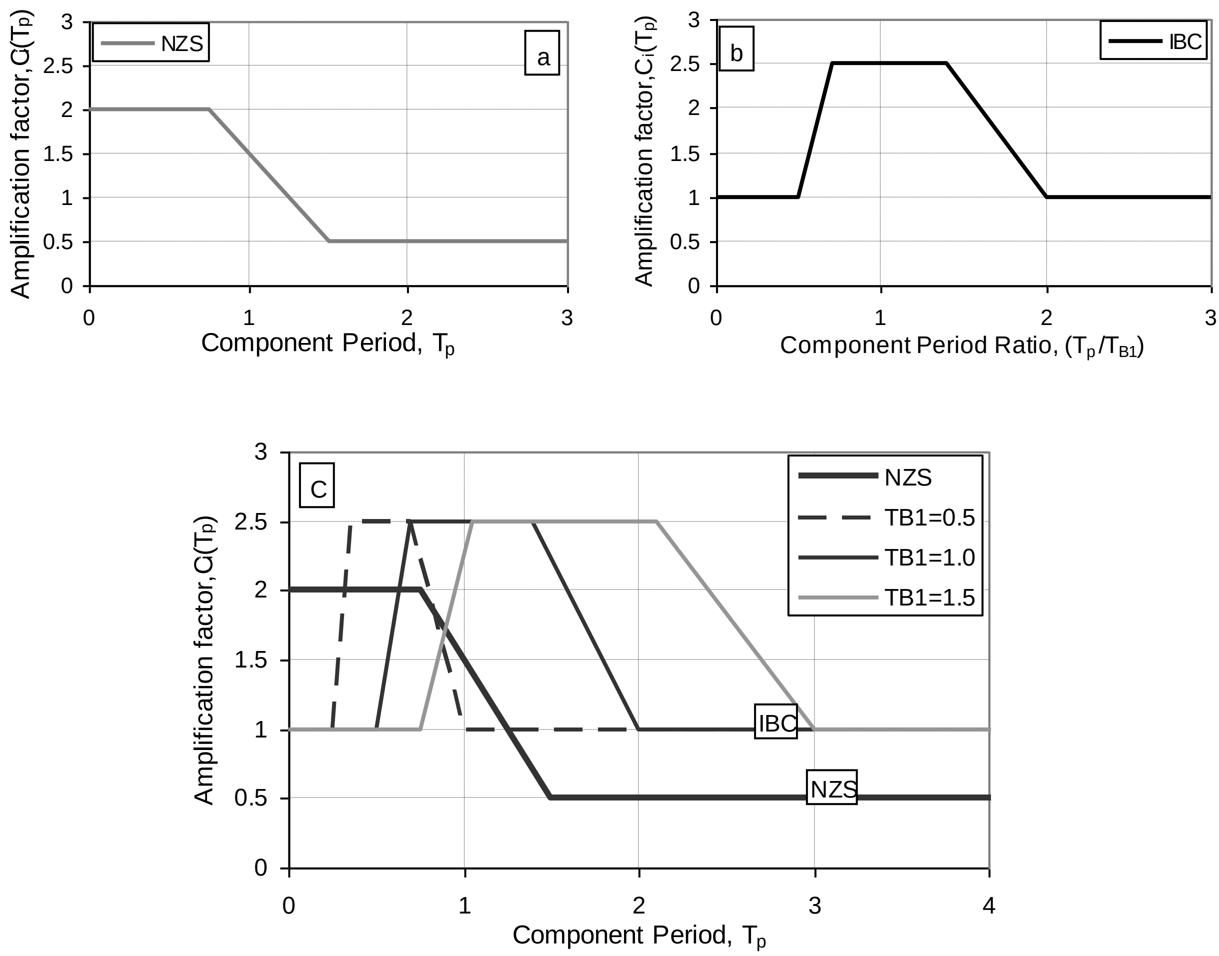


***Figure 2:** Component amplification factor: (a) NZS provisions independent of T_{B1}; (b) IBC provisions; (c) comparison of NZS provisions with IBC provisions for T_{B1}= 0.5s, 1.0s, and 1.5s.*

Research studies have shown that current provisions as in IBC do not represent the 'actual' variation of floor accelerations up the height of the building, rather that the floor accelerations are closer to a constant value due to the combined effect of the contributions of higher modes and non-linear behaviour of the building [9, 13, 19, 20].

Figure 2a shows the component amplification factor from NZS provisions, which is expressed as a function of the component period T_p. The amplification factor is constant at 2 for all components with periods less than 0.75, decreases linearly to 0.5 at 1.5s, and remains at 0.5 for longer periods. In contrast, SEI/ASCE-02 and IBC provisions relate the factor to the ratio of component period to building period, T_p/T_{B1}, and attempt to capture the amplification near the region when T_p/T_{B1} equals 1 (Figure 2b). For comparison purpose, the component amplification factor suggested by IBC is plotted for 3 different values of building period (T_{B1}= 0.5s, 1.0s, and 1.5s) along with NZS provision which is independent of building period (Ref. Figure 2c). It can be noted that: (i) IBC provisions do not recommend any amplification for components with very short periods (representative of rigid components), whereas NZS specifies an amplification factor of 2; and (ii) Short period components experience higher amplification in low-rise and stiff buildings (say, T_{B1} = 0.5s) than in high-rise and flexible buildings (say, T_{B1} = 1.5s). Some studies have indicated that the amplification of floor acceleration is influenced by parameters such as ratio of component period to building period and inherent damping of equipment [21].

Before proceeding with the current investigation, brief comment is made on the development of the NZS 1170.5 provisions [14]. The current provisions are based on an extensive study by Shelton [22]. In that study, a suite of buildings representing 3, 10 and 20 storeys with two structural materials (reinforced concrete and steel) and of limited and high ductility, located in low and high seismicity areas were considered. Three dimensional models were prepared for 31 cases; however each was subjected to only three ground motion records (2 components each) which were scaled to correspond to ULS design level only. The design recommendations were made by 'enveloping' the analysis results which formed the basis of the equations used to develop Figure 1 (as FHC) and Figure 2a (as CAF). Therefore, the current recommendations as in NZS 1170.5:2004 are based solely on ULS responses as the study did not include analyses for any other intensities of shaking. However, it is interesting to recall that NZS 4203:1992 [23] provisions expressed floor accelerations as a function of period ratio.

BUILDING MODELS

Building models considered in the present study represent ductile 3 storey and 10 storey buildings with two structural systems: (i) reinforced concrete moment-resisting frame (RC); and (ii) steel eccentrically braced frame (ST), with a ductility of 6 and 5 respectively. In this study, the buildings were identified as 3 and 10 storey buildings excluding the roof level, similar to the practice adopted in earlier studies [22]. With reference to the HAZUS99 [24], 3 and 10 storey buildings were considered to be representative of low-rise and high-rise building respectively. The buildings were designed following NZS 1170.5:2004, for shallow soil sites in a high seismicity area (Wellington) using the capacity design approach to achieve a target inter-storey drift ratio of 2.5%, which is the maximum limit permitted by the standard. However, certain minimum requirements from materials standards with regard to the member sizes precluded the achievement of this limit and also controlled the strength and the ductility experienced. For example, in the case of a steel 3-storey building (ST3), the stress limits in the 'links' controlled the design resulting in a stiffer frame than for the other cases. However, the influence of actual ductility experienced on the amplification of floor acceleration was not investigated within the scope of the present study.

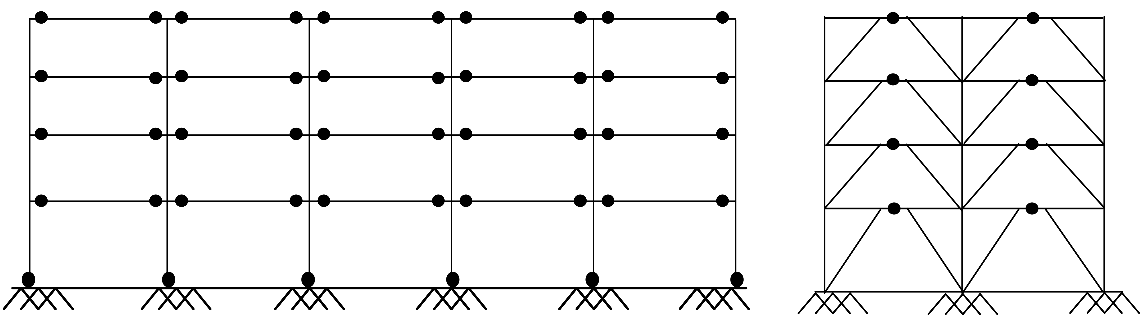
The nonlinear dynamic simulations were performed with 2-dimensional models of the buildings in SAP2000 [25]. Cyclic inelastic deformations were modelled using nonlinear link elements. These elements were included in beams near the column faces for RC buildings and in the links of the frames in ST buildings. In RC buildings the column elements in the ground floor were modelled with fibre hinges to account for axial load-moment-interaction. P-Delta effects were considered in the analyses.

The height of the ground story was 4.5 m and other storeys were 3.65 m high. The bay length was 7.5 m for the RC frame and 8.5 m for the ST frame. The seismic mass on all the floors was 5,450 kN, with 5,150 kN for the roof. Typical configurations of the RC and steel low-rise buildings are shown in Figure 3. The building modal periods are given in Table 1.

GROUND MOTIONS

Two suites of ground motions were considered with intensity corresponding to annual probabilities of exceedence of 10% in 50 years (referred to as ULS) and 63% in 50 years (referred to as SLS). The buildings were analysed mainly for ground motions generated by shallow crustal events. Five horizontal components from five records were selected and scaled to represent the likely ground motions for the 500-year return period (to simulate ULS condition). The other components were not used because the match to the corresponding code design spectra was not as good as for those selected. Eight horizontal components from four records from shallow crustal earthquakes were selected to represent possible ground motion for a return period of 50 years (to simulate SLS conditions). The C(0) values are obtained from NZS 1170.5:2004 using return period factors of 1.0 and 0.35 for ULS and SLS conditions respectively.

The ranges of magnitude and source distance for the records were selected to be the best match to those of the deaggregation results from the probabilistic seismic hazard analyses. The records were scaled to match the design spectra for site class C according to the current design standard (NZS 1170.5: 2004). The earthquake magnitudes and the closest source distances to the rupture planes for crustal events are given in Table 2. The selected records met the NZS requirement for scaling factor, k_1, i.e. ($0.33< k_1< 3.0$), except for the ARC2 record (with a scale factor of 3.5). This record was chosen because of its magnitude level that is similar to the possible Wellington fault event. Note that only the LUC1 record from the 1992 Landers earthquake contains forward directivity pulse. However, all the records selected were matched with the design spectra enhanced by the code near-source factor that enables to include the effect of forward directivity effect. In the later part of this paper, the effect of ground motions from subduction slab events on floor response will be discussed.



- Location where non-linearity is modelled

Figure 3: Typical 2-dimensional models for 3-storey RC and ST buildings.

Table 1. Modal Periods of the Buildings.

Buildings	First Modal Period (FM)	Second Modal Period (SM)
RC3	1.1	0.3
ST3	0.7	0.25
RC10	2.3	0.8
ST10	1.7	0.6

Table 2. Information on the strong-motion records for crustal events selected for the present study.

Record Name	Comp.	Station Name	Earthquake Name	M_W	R (km)
500 year return period (5 components)					
ARC2	EW	Arcelik	1999 Kocaeli, Turkey	7.5	14
DUZ2	270	Duzce	1999 Kocaeli, Turkey	7.5	15
ELC2	270	El Centro	1940 El Centro	7.0	7
LUC1	260	Lucern	1992 Landers	7.3	2
TAB2	NS	Tabas	1978 Tabas, Iran	7.4	2
50 year return period (4 records with 2 components)					
A-IV1/2	90/360	Wildlife Liquef. Array	1987 Superstition Hills	6.2	18
B-BRA1/2	225/315	Brawley Airport	1987 Superstition Hills	5.8	17
B-LAD1/2	180/270	Bishop - LADWP	1986 Chalfant Valley	5.8	23
H-PTS1/2	225/315	Parachute Test Site	1979 Imperial Valley	6.5	12

FLOOR HEIGHT COEFICIENT

The floor height coefficient is defined as the peak floor acceleration normalised by the site hazard coefficient, C(0), at each floor levels up the building height. Peak floor acceleration is defined as the maximum acceleration of the floor from time history response. The design of a rigid component with a period close to zero is based on the magnitude of peak floor acceleration. The median and 84-percentile values of FHC are plotted under ULS and SLS conditions as shown in Figure 4. It can be noted that the provisions from NZ standards are highly conservative compared to the analytical results. Also, the FHC is almost constant for all floor levels for both SLS and ULS ground motions, and significantly differs from the first structural modal shape. The reason may be that PFA is associated with high-frequency content (possibly over 3 Hz) of the floor acceleration time history where as the first mode periods of the buildings considered in this study are long; and hence amplification of response by the building at frequencies greater than 3 Hz may not be large both under ULS and SLS ground motions. The analysis results imply that the design force for a given rigid component remains the same irrespective of its location with respect to the height of the building whereas NZS recommends an increasing design force as the floor height increases.

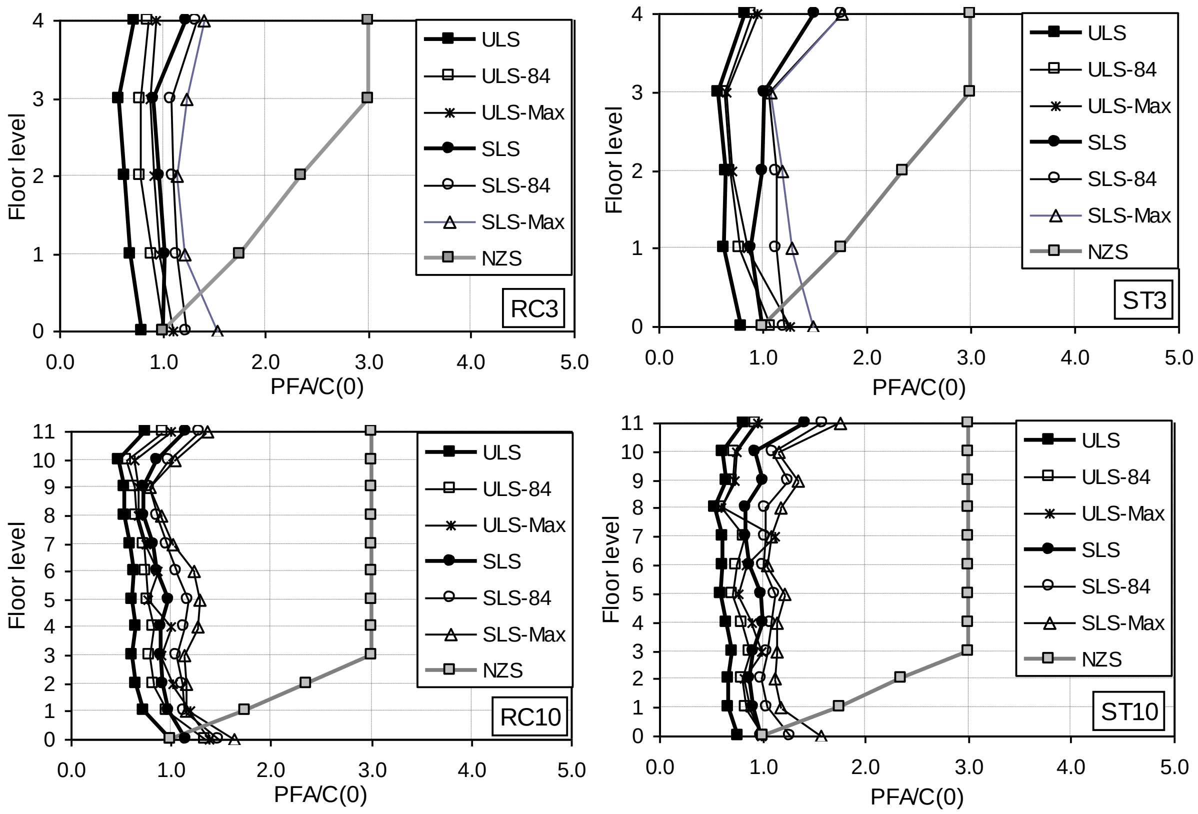


Figure 4: Floor height coefficient distributions for ULS and SLS intensities.

It should also be noted that FHC values resulting from the analyses with the ULS records are less than those from the SLS records, a possible result of the significant nonlinear response of the structure under ULS earthquake loading. The slight increase in FHC at roof level is likely the result of high frequency modes. It is worth noting that at ground floor level PFA/C(0) is not equal to 1.0. The median of peak accelerations of the ground floor is equal to the median of peak ground acceleration of the chosen earthquake records (considering the base of the structure is fixed and no soil-structure interaction is included). The selected records are matched to the design spectrum only within a certain range of periods related to the building period, T_{B1} (i.e. 0.4 T_{B1} to 1.3 T_{B1}) and not at the zero period, thereby: (i) the scaled earthquake records have different PGA values; and (ii) they do not match the C(0) value of the design spectrum. Hence, PFA/C(0) at ground floor level is not necessarily equal to 1.0.

The almost constant PFA distribution up the building height is supported by recorded building response. Two sets of records from a building array in Gisborne in NZ, i.e. the Gisborne Post Office which is a reinforced concrete moment resisting frame building with 6 floors, have shown nearly the same PFAs at roof, and 2nd Floor and the free-field records for the Ormand earthquake in 1993 [26]. As a side issue, the PFAs at the basement of this building are always considerably less than those at the free-field, the 2nd floor and the roof as a result of kinematic soil-structure interaction.

The floor height coefficient in the current design code is based on the results from the Shelton study [22], where the distribution is largely the envelope of maximum PFAs computed from time history analyses. The results presented in the present study include median, 84- percentile and maximum values.

The large difference between the code provision and the results presented here could be because the number of records used in both the Shelton [22] study and the present study are small. When the number of records is small, the envelope values will be affected very strongly by the choice of selected records, even though each selected record is matched to the design spectra in a period range around the building natural period. This is because, the envelope values can be dictated by the values of floor acceleration spectra at a spectral period outside the matching period range which is not controlled within the scaling range. So, the envelope values could be very sensitive to the response of the individual record. However the median and 84-percentile values are much less sensitive to the record selections than the envelope values.

While from a design safety point of view selecting envelope values for PFA may be preferred, the large sensitivity to the record selection may lead to over-conservative estimates (or under-estimates on rare occasions) of PFAs. On the other hand, results that are statistically derived may be considered as more representative values than those obtained based on envelopes of maximum values.

COMPONENT AMPLIFICATION FACTOR

The component amplification factors, as given in Equation 1, are computed as the ratio of the spectral response of the floor normalised to the respective peak (zero period) floor acceleration. They are presented in Figure 5 for a low-rise concrete and a high-rise steel structure. The analytical median responses for both ULS and SLS conditions are compared with NZS envelope (see Figure 2a). Since the response spectra is normalised with PFA, S_ac/PFA is 1.0 at zero period. Under SLS conditions, the NZS envelopes are exceeded by a large margin at both the first and higher mode periods. Under ULS conditions, although the NZS envelope is comparable at higher mode periods at lower storeys (particularly for the high-rise structure), it is exceeded by a large margin at the first mode periods for both low and high-rise structures. These observations underpin the importance of building period to component amplification.

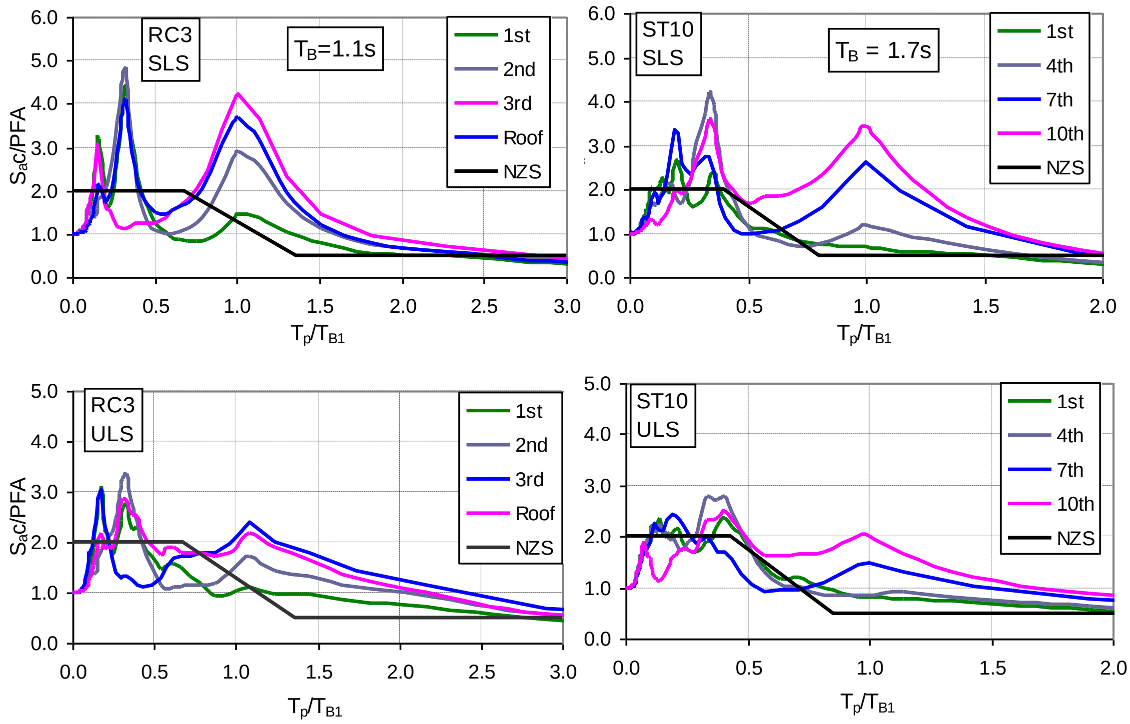


Figure 5: ***Component amplification factor for low and high-rise buildings under SLS and ULS conditions – the analytical CAF exceeds the NZS recommended envelope.***

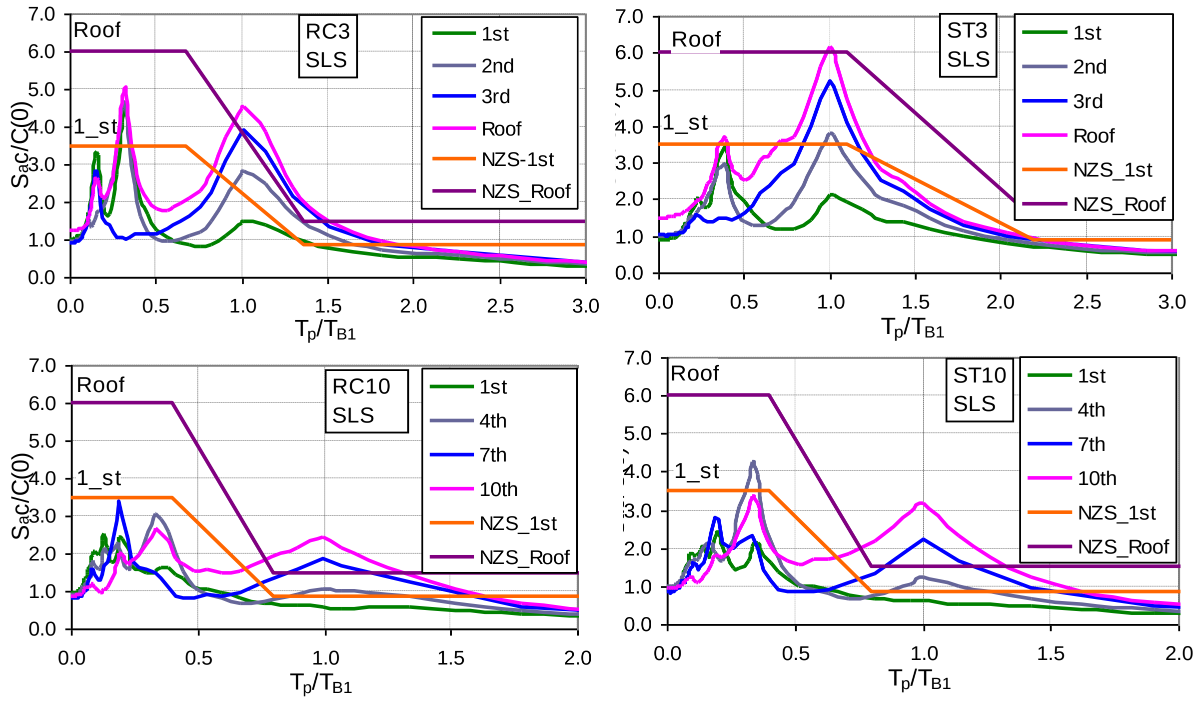


Figure 6: Prominent peaks at modal periods for SLS intensities.

From the above analysis results on floor height coefficient and component amplification factor, it appears that: (i) the NZS floor height coefficient distribution is highly conservative; and (ii) the NZS spectral shape representing component amplification factor is un-conservative. Note that in Equation 1, the total design demand is obtained as the combination of FHC and CAF, so the total analytical spectral demand may or may not exceed NZS provisions, for a specific T_p/T_{B1} ratio. In the following sections, the total analytical spectral demands are compared with the design envelopes as per NZS provisions for the buildings considered under ULS and SLS conditions.

SPECTRAL RESPONSE OF COMPONENTS

In this section, the amplification of the spectral response of components (S_ac) is expressed with respect to the ground intensity parameter, C(0) for SLS and ULS conditions. Median spectral responses are plotted in Figure 6 for low-rise buildings. The component periods are normalized by the first modal period of the buildings to enable clear identification of 'resonance' effects. These effects are evident under SLS conditions, where the amplification ratio increases significantly with increasing floor level at T_p/T_{B1}= 1, more so than under ULS conditions. In this section, focus is given to : (i) the variation of peak floor acceleration (at T_p/T_{B1}= 0) (ii) response amplification at SLS intensities of shaking (Figure 6) and (iii) response amplification at ULS intensities of shaking (Figure 8).

Comparison of response spectra of all the floors with respect to C(0) in Figure 6 shows that the floor accelerations for components with periods very close to zero are not amplified up the height of the building. In other words, approximately constant peak floor acceleration is experienced up the height of the building (as seen in Figure 4). From this limited study it appears that the rigid components (with periods less than 0.06s as recommended by FEMA 450 [27]) may adequately be designed to lesser demand levels than those currently required by NZS provisions. The current NZS provisions appear to be very conservative by requiring a spectral amplification factor of 2 for rigid components with periods close to zero.

Figure 6 shows amplifications of floor response spectra under SLS intensities of shaking. The general observation is that the amplifications do not follow similar pattern for all the floors. 'Resonance' effects are prominent at fundamental and higher mode periods. At some frequencies, near higher modes, the amplifications exceed those at first mode, in particular at lower stories. It shows that possibly the short period components will experience higher demands when located in lower storeys of frame buildings. The analytical responses are compared with design envelopes from NZS provisions selectively at two floor levels namely, 1^{st} and top floor levels. The design envelopes represent only elastic design forces, and linear response is expected under SLS conditions. Amplifications in first mode and one or more higher modes are clear. The median responses plotted in Figure 6 are close to or above the design envelope, and hence the 84-percentile responses would exceed the design envelope significantly at the building natural period.

Note that the normalized floor response spectra at the building natural period increase with increasing height, similar to the PFA distribution up the building height in the current New Zealand code. On the other hand, the ratios between floor response spectra and the ground-motion spectra at the building natural period can be very similar for buildings with different natural period but with similar damping ratios when these buildings do not develop large nonlinear response, as in the case for SLS conditions. Figure 7(a) shows the ratios between floor response spectra and ground response spectra up the normalized building height for four structures under SLS conditions. The spectral ratios are similar for all buildings at a given height ratio (h/H) at least up to a height ratio of 0.7. The spectral ratios at the top floor of the buildings vary between 4 and 6 for different buildings. Let us assume that the response spectral ratio, S_R is expressed as a function of building period, T_{B1} and height ratio, h/H as:

$$S_R(T_{B1}, h/H) = \frac{S_{aC}(T_{B1}, h/H)}{S_g(T_{B1})} \quad (2)$$

where H is the total building height, h is the height of a given floor, S_{aC} is the floor response spectra and S_g is response spectra of the ground motions. The normalized floor response spectra with respect to C(0) can be expressed as

$$\frac{S_{aC}(T_{B1}, h/H)}{C(0)} = S_R(T_{B1}, h/H)\frac{S_g(T_{B1})}{C(0)} \quad (3)$$

Note that S_R is approximately constant at the building natural period and for a given building height ratio and this leads to the conclusion that the normalized floor spectra for buildings with different natural periods are proportional to the normalized ground motion spectra $S_g/C(0)$. For two buildings with 2 different natural periods, $T_{B1,1}$ for the first building and $T_{B1,2}$ for the second building, the ratios of the floor response spectra for the two buildings can be written as,

$$\frac{S_{aC}(T_{B1,1}, h/H)}{S_{aC}(T_{B1,2}, h/H)} = \frac{S_g(T_{B1,1})}{S_g(T_{B1,2})} \quad (4)$$

Although the selected strong motion records are scaled to match the design spectra at different period ranges for buildings with different natural periods, the normalized ground motion spectra, as given in the right hand side of Equation 3, have generally similar shape as a function of period, in an average sense, as shown in Figure 7(b), even though matching the design spectra was carried out at different period band for buildings with different natural period. The relative values of the normalized floor spectra at building natural periods for all the buildings (RC3, ST3, RC10 and ST10) in Figure 6 can be worked out from Equation 4 and Figure 7(b) with an acceptable accuracy.

From the above discussion it can be argued that for any generic building, the normalised floor response (S_ac/C(0)) of components with period equal to the fundamental period of the building can be extrapolated using the results from Figures 6 and 7 together with Equation 3. For example, at third floor level (roughly about h/H = 0.7), the peak normalized floor spectra is 5 for the ST3 building with a natural period of 0.7s. To develop the amplification for a building and component with a natural period of 0.4s, the spectral ratio between 0.4s and 0.7s from Figure 7(b) is about 1.5. Using Equation 4, the normalized floor response spectra at 0.7H level for a building with a natural period of 0.4s would be about 7.5, 25% higher than the values given by the NZ provision. However, for components at roof level, the response could vary between 0.8 to 1.2 times the average responses, due to variations because of the higher mode effects. From the results presented here, it appears that a stiffer structure with a period shorter than 0.7s, (but flexible enough to amplify, i.e. with T_{B1} at least greater than 0.2s) is likely to exhibit larger amplification, exceeding the current design envelope by a large margin.

The constant response spectral ratios between floor response and the input ground motion at a given building height ratio suggest that the findings of the present study under SLS conditions could be applicable to other buildings that have the same fundamental period, irrespective of the number of storeys. Nevertheless, further studies on buildings with shorter periods will be helpful to substantiate this.

In this paragraph, the importance of expressing spectral amplification as a function of component period and building period is discussed. Note that the NZS envelopes in Figure 6 represent the variation of design demand for components with a range of periods of vibration for the particular floor. Because the design envelope in NZS 4219:2009 [28] does not account for building natural period, the two corner points of the envelope appear at different normalized period for different buildings. The design envelope covers the SLS floor spectra peak very well for the ST3 buildings with a relatively short natural period of 0.7s in Figure 6, especially the sloped part of the envelope. The envelopes miss the peak normalized spectra associated with the building period for other structures with moderate and long periods. For example, the envelope at the roof level for the RC3 building (T_{B1}= 1.1s) does not cover the normalized floor response spectra at T_p/T_{B1} range between 1 and 1.5. The design envelopes at roof level for the ST10 and RC10 buildings are considerably lower than the computed floor spectra within a range of 0.8-1.2 for T_p/T_{B1}. A design envelope as a function of building natural period will help eliminate this discrepancy. The design envelope is very conservative for non-structural components with a period considerably shorter than the building natural period.

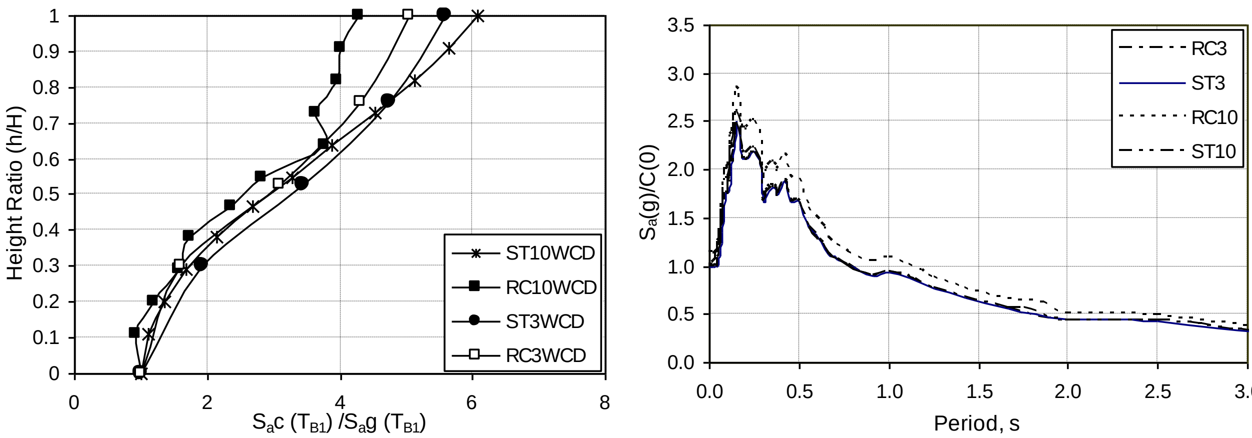


Figure 7: ***The average floor spectral ratios for 4 structures (left) and the average normalized spectra of ground motions.***

Figure 8 shows the spectral amplifications of floor response under ULS conditions for both low and high rise buildings. A prominent feature for ULS level intensities is the marked reduction in the normalized floor response spectra in a spectral period range around the building natural period compared with those in Figure 6 for the SLS intensity level, presumably a result of nonlinear response under strong shaking (ULS) levels. The reduction factor in the peak floor spectra close to the building natural period comparing SLS to ULS shaking intensity is about 2 for the ST3 building with a natural period of 0.7s, and about 2.5 for the ST10 buildings with a natural period of 1.7s. The reduction factor for RC3 building with a period of 1.1 seconds is about 2.6 and is 5 for the RC10 building with a natural period of 2.3s. The very large reduction in the RC10 building may be a result of lengthening of the equivalent building period beyond that of the other buildings. The reduction factors for the two RC buildings are larger than for the two steel buildings, suggesting that higher ductilities are developed in the reinforced concrete buildings than in the steel buildings. The normalized floor response spectra at the building natural periods still follow a similar pattern as in the SLS case: i.e. the normalized spectra at the building natural period and at roof level decrease with increasing building natural period.

In general the amplification peaks are more pronounced at short component periods than at long component periods. Notably, under ULS level of intensities, the responses are 'softened' near the fundamental periods of the buildings due to their non-linear behaviour.

Figure 8 shows that the design envelope is adequate to cover the median values at long periods for all four buildings but exceed the median values by a large margin for component periods less than half of the building natural period.

Under ULS conditions some component categories are permitted to undergo inelastic behaviour. In such situations, NZS provisions recommend reduced design forces to account for the 'ductile performance' of seismic restraints of the system. NZS 4219:2009 [28] suggests 'performance factors' to reduce the design loads for various types of seismic restraints. For example, a value of 0.55 is recommended for a typical floor mounted component with a 'ductile base fixing'. For illustrative purposes, NZS design envelopes are drawn in Figure 8, assuming that the components are provided with such restraints that a performance factor of 0.55 can be used. These envelopes are drawn for components located at the roof level only. In Figure 8, the 'elastic' envelope represents the elastic design forces and the 'inelastic' envelope represents the reduced design forces after accounting for performance factor. The advantage of reducing the design forces by using a 'performance factor' is clear for short period components. Note that the normalized floor spectra presented in the present study are 5% damped elastic spectra for non-structural components. The authors expect that the inelastic floor spectra corresponding to a reduction factor of 0.55 at most periods would be considerably less than the elastic floor spectra presented in Figure 8. This means that even, if the in-elastic demand specified by NZ code provisions is less than the elastic floor spectra, the code provision is not necessarily unsafe.

Note that floor response spectra are constructed for 5% critical damping in this study. The response will be higher if the component exhibits damping less than 5%, which may well occur under low intensity shaking. Therefore, the actual demand for such components with less damping under SLS intensities might well exceed the design envelope specified by NZS provisions, especially when the component period is close to one of the modal periods of the building.

In summary, the demands on rigid components with periods close to zero are likely to be much less than design levels required by current NZS provisions. This conservatism is countered under SLS conditions by component amplification higher than specified by NZS such that the demand on flexible components under low intensity motions is of similar magnitude to the current NZS provisions. The spectral amplifications under ULS conditions, however, are markedly smaller than those under SLS conditions within a range of T_p/T_{B1}= 0.2 to 1.5, a possible result of the nonlinear response developed in the building under the ULS conditions. Also, amplification does not have dominant peaks at T_p/T_{B1}= 1 under ULS conditions and so NZS provisions tend to be overly conservative in this range. Invariably amplification is observed near the higher (second and third) modal periods both in ULS and SLS conditions.

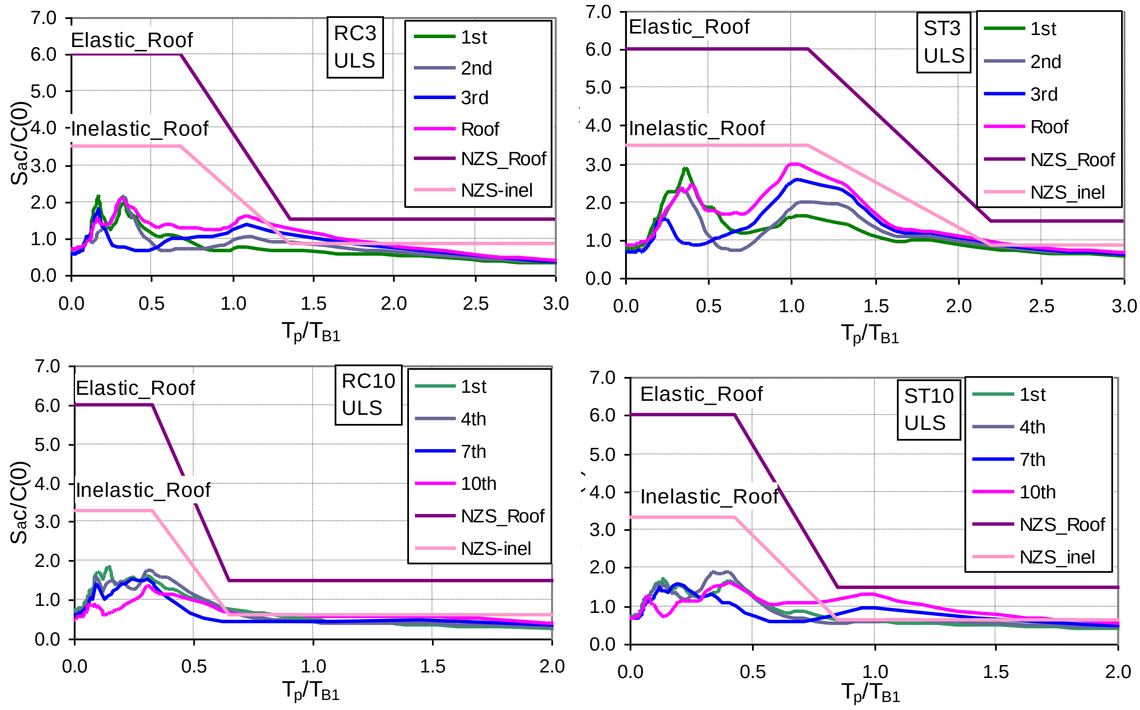


Figure 8: Less spectral amplification near fundamental period range for ULS intensities.

ACCELERATION DEMANDS

In this section, the acceleration demands on components with short and long-period of vibration are discussed. The acceleration demands from numerical analyses are compared with design acceleration demands required by the NZ standards. NZS 1170.5:2004 specifies maximum spectral amplifications for components with periods from zero to 0.75s so components with periods less than or equal to 0.75s are considered here to be short-period components and others as long-period components. Since resonance in amplification is observed at building modal periods, the design demands are computed for long period components (with a period assumed equal to the first mode (FM)) and for short period components (with a period assumed equal to the second mode (SM)) of respective buildings.

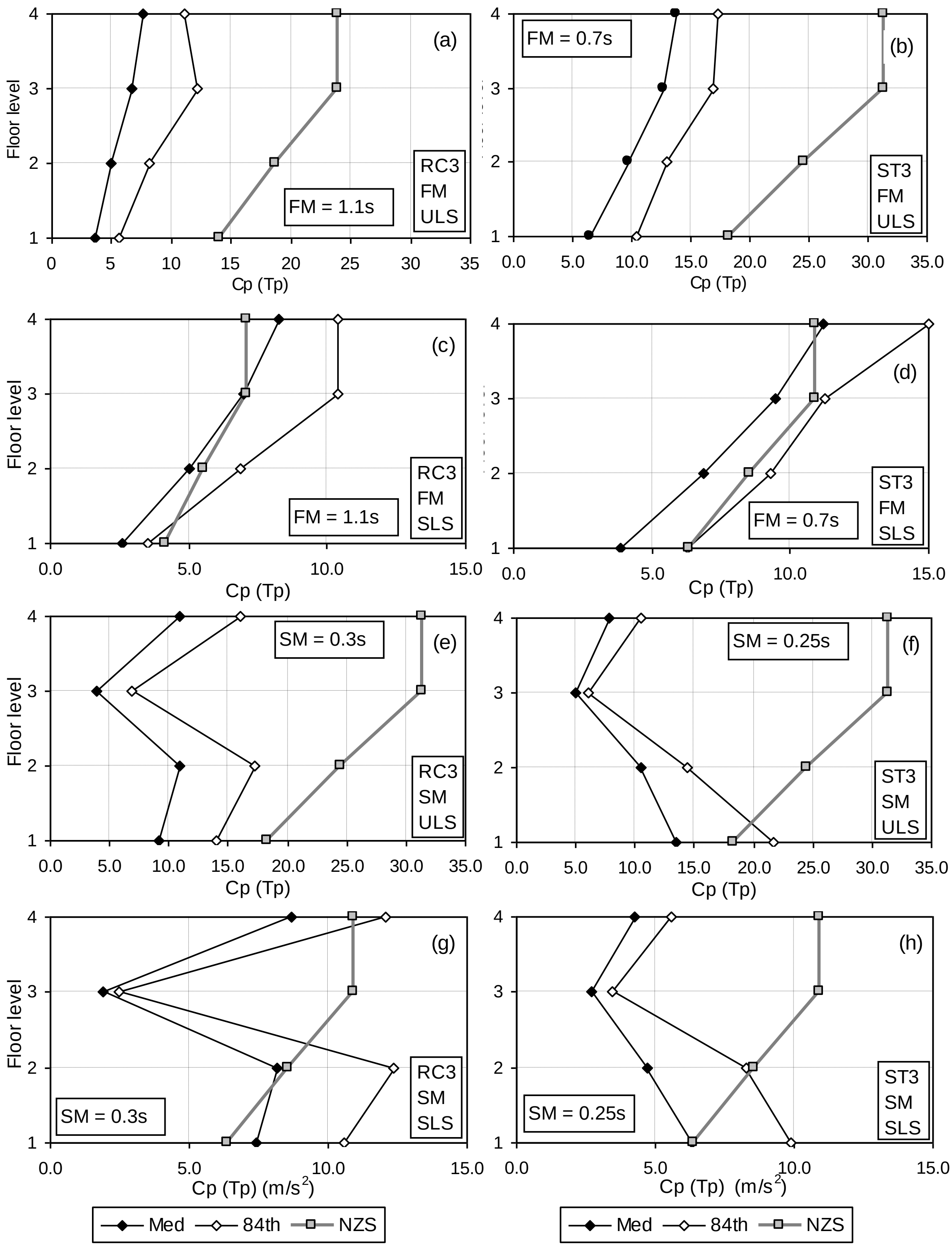


Figure 9: Acceleration demands (m/s²) on components in low-rise buildings under ULS and SLS intensities.

The design acceleration demands for short and long period components are obtained as per NZS 1170.5:2004 using Equation 1. The C(0) values representing site response spectra at zero period are 0.53g for the 500-year return period and 0.19g for the 50-year return period. The values of C_{Hi} and $C_i(T_p)$ are obtained from Figure 1 and Figure 2(a) respectively; i.e. combining the effects of peak floor acceleration and spectral amplification of the component with the given period.

The numerical results of acceleration demands for short and long period components for low-rise and high-rise buildings are presented and compared with design envelopes from NZS in Figures 9 and 10. Note that the envelopes from Equation 1 represent the distribution of maximum elastic design force corresponding to each floor level up the height of the building for components with a specified vibration period (equal to FM or SM). The median and 84-percentile values corresponding to first mode (FM) and second mode (SM) periods are plotted.

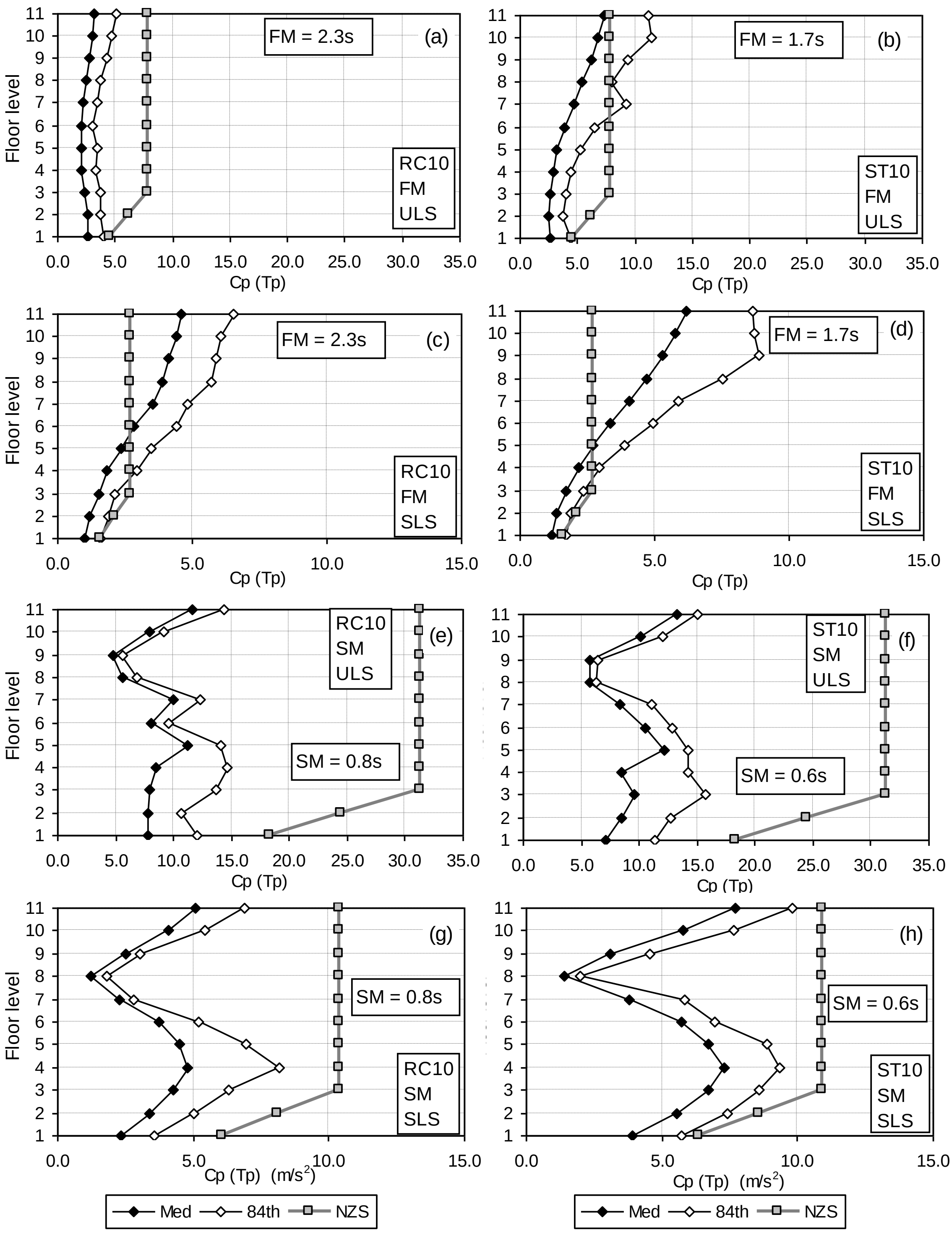


Figure 10: Acceleration demands (m/s²) on components in high-rise buildings under ULS and SLS intensities.

Low-rise buildings

Figures 9(a), (e) and (f) show that the computed demands (median and 84-percentile) are less than the NZS envelope for both categories of short- and long-period components (SM and FM) in ULS conditions as a result of the lesser amplification observed in ULS conditions, with one exception in Figure 9(f) where the 84-percentile demands exceed NZS envelopes marginally in floor level 1 for short-period components. Figures 9(c) and 9(d) show that the median demand at the fundamental period exceeds the design envelope at roof level for both low rise structures, for SLS shaking intensities and the 84-percentile demand exceeds the design envelope for the top 3 levels.

If the natural period of a building (in this case, ST3 building) is shorter than 0.7s, we expect that median and 84-percentile demands for components with periods closer to the building natural period may exceed the NZ design envelope considerably.

High-rise buildings

Figure 10 compares the computed median and 84-percentile spectral demands at the first and second modal periods with NZS provisions for high-rise buildings under ULS and SLS conditions. At ULS shaking intensity, the spectral amplification at T_p/T_{B1}= 1.0, is very low in the RC10 building (ref. Figure 8) and so both median and 84-percentile spectral demands at the building natural period (FM) are well within the NZS envelope representing the adequacy of NZS design provisions for long period components (Figure 10a). However, the ST10 building has moderate amplification such that the median demands at the building natural period are close to the NZS design envelope at upper floors and the 84-percentile demands exceed the design envelope. As discussed previously, a possible reason for very low and moderate amplifications is related to the extent of non-linearity experienced at the fundamental mode in the respective buildings.

Under SLS condition, the median and the 84-prercentile spectral demands at the building natural periods (FM) significantly exceed the NZS design envelope at floors over about 1/3 of the building height from the ground level. Two possible reasons for the large exceedance, other than a reduced non-linear effect can be given: (i) the spectral amplification as per the NZS provisions is building period independent thereby not accounting for possible amplifications at larger natural period corresponding to high-rise buildings; (ii) the spectral amplification increases up the height of the building, however, NZS provisions defines a constant spectral shape factor at all the floor levels.

The demands on short-period components in SLS and ULS (SM) are always below the NZS provisions. Under the SLS intensity of shaking, the response of high-rise buildings is essentially dominated by the fundamental mode of vibration resulting in amplification of acceleration in upper floors, thereby increasing the demands on long period components rather on short period components.

The adequacy of NZ provisions compared with analytical demands for the categories of components in low and high-rise buildings under ULS and SLS design levels are summarised in Table 3.

Table 3. Adequacy of NZS provisions for short and long period components.

Component category	Low-rise building		High-rise building	
	ULS	SLS	ULS	SLS
Short period	Adequate	Marginal- lower stories	Adequate	Adequate
Long period	Adequate	Marginal- upper stories	Adequate	Inadequate in upper stories

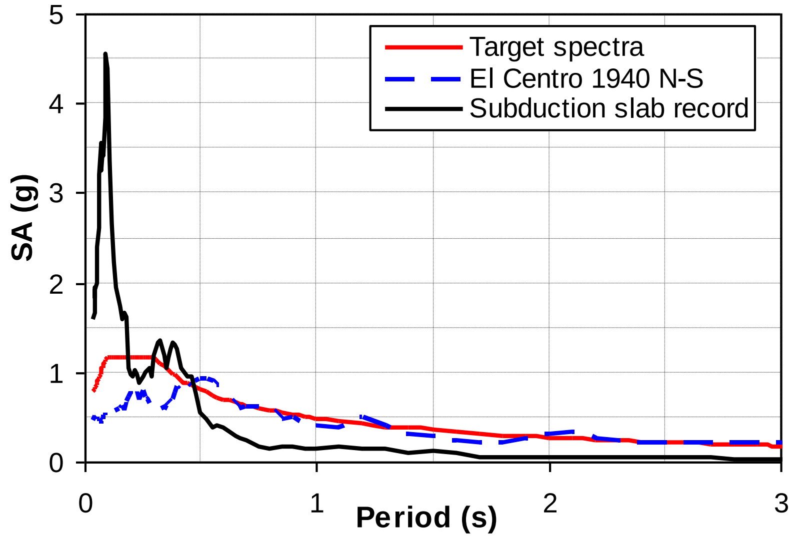


***Figure 11:** A clear illustration of record scaling and the different frequency contents between a shallow crustal record (El Centro 1940) and a subduction intra-slab record.*

GROUND MOTIONS FROM SUBDUCTON SLAB EVENT

A brief study was carried out to illustrate the effect of ground motions from a different type of earthquake, namely a subduction slab event, which was intended to represent a possible earthquake from the subduction zone beneath Wellington. This was intended to assess the effect of frequency content of the input motion on floor response spectra. Table 4 shows the list of records considered for both ULS and SLS intensities of shaking. For this exercise, the evaluation was restricted to the response of 3-storey reinforced concrete frame (RC3).

Earthquake motions from subduction slab events produce very high short period ground motion compared with shallow crustal events (Figure 11). Note that ground motion selection procedure as per NZS1170.5:2004 suggests that only strong-motion records that match the design spectra in a broad period range are appropriate to be used for analyses. As a result, selected records tend to have very similar spectral shape and the effect of frequency content is diminished. If the scaling procedure as given in NZS1170.5 is followed, the short period spectra for the subduction slab earthquake would be unrealistically high; and so matching to the design spectra is done only for short period range (0.4s) for slab event records (often referred to as intra-slab earthquake).

The intention is to show typical response characteristics for slab events, and compare them with the envelope specified by NZS provisions. Because of the very small number of records, the median and the 84-percentile values are very similar and hence only the former are shown in Figures 12 and 13 to illustrate peak floor acceleration and spectral amplification with regard to C(0) respectively. Similar to the crustal events, the variation of peak floor acceleration up the height of the building is small for slab events as shown in Figure 12.

Figure 13 shows floor spectral amplification under slab events for both ULS and SLS intensities. A few prominent differences are noted in the floor response spectra between subduction slab earthquakes and crustal events.

Table 4. Information on the strong-motion records for subduction intra-slab events selected for the present study.

Record Name	Comp.	Station Name	Earthquake Name	M_W	R (km)
500 year return period					
K0351	NS	IWT007	26-05-2003 Japan	7.0	71
K0352	EW	IWT007	26-05-2003 Japan	7.0	71
50 year return period					
KLA1/2	NS/EW	HRS014	24-03-2001 Japan	6.8	63
KMN1/2	NS/EW	EHM003	24-03-2001 Japan	6.8	63

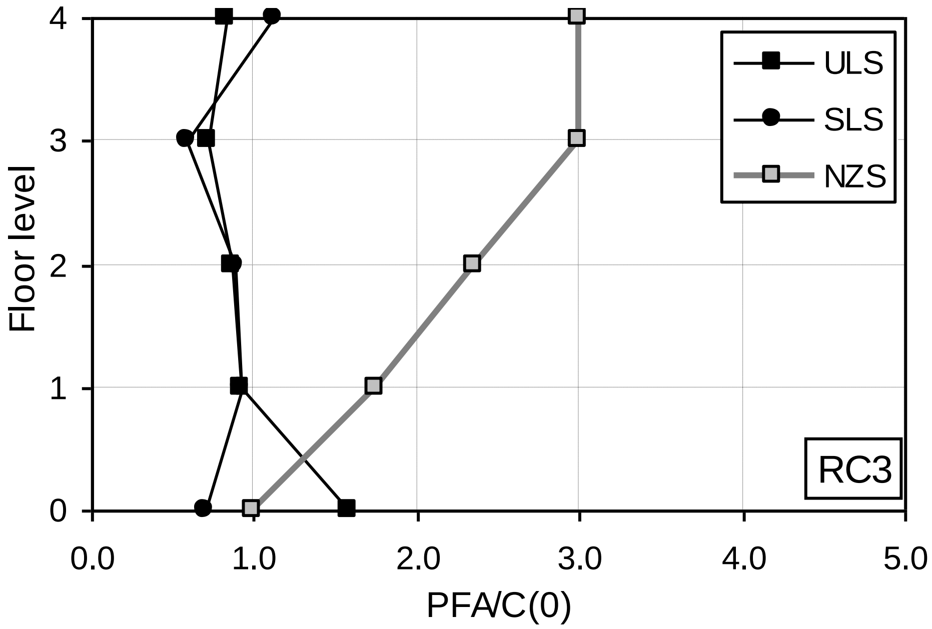


Figure 12: Floor height coefficient distributions under slab events.

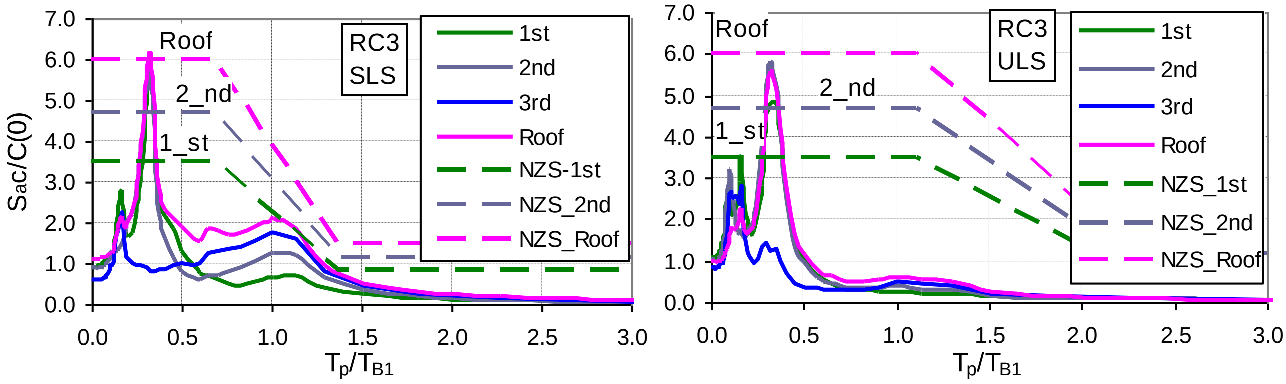


Figure 13: Spectral amplification under slab events.

Under SLS conditions, peak normalized floor response spectra at the building natural period are less than half of those for crustal earthquakes (as shown in Figure 6) at all floor levels. This is caused by the differing frequency contents of the input ground motions. As shown in Figure 11, the spectral acceleration at the building natural period (1.1s) for the slab record is less than 0.2g while the design spectral acceleration is 0.45g at this period. The ratio between the design spectral acceleration and the average spectral acceleration of the slab earthquake records at the building natural period is approximately 2.3 and the ratio between the normalized floor spectra at the building natural period for crustal earthquakes and the slab earthquakes is also very close to 2.3. The peak floor spectra at the second modal period of the building for slab events are higher than those in Figure 6, although by only about 20%. The NZ design envelope is considerably larger than the computed spectra at spectral periods greater than the second modal period of the building at all floor levels. However, at the second modal period of the building, the normalized floor spectra exceed the NZ design envelope at first and second floors and just exceed it at roof level. Note that there is very little amplification at third floor level as this is close to a nodal point (zero displacement) of the second mode.

Under ULS shaking intensities, the normalized floor accelerations show pronounced peaks at second mode period but little or no amplification at building's first mode period, the latter effect very similar to crustal events. The NZS design envelope is conservative at all levels for spectral periods exceeding 0.4 times the building natural period.

Near the fundamental period of the structure, the floor response spectra are considerably lower than the spectral response from shallow crustal records in ULS but the design envelope is marginal in SLS conditions. A possible reason is that high frequency contents of the earthquake do not contribute to the amplification at the longer periods (i.e. closer to the fundamental period of the building, which is 1.1s).

In conclusion, the effect of the high frequency content typical of slab events is clearly pronounced in the form of high peaks near shorter periods rather than at longer periods. So, under ULS intensity level, a slab event will prove much more damaging than a crustal event, especially in the performance of short period components, if designed according to current NZS provisions. In SLS conditions, there is a likelihood of such components sustaining damage if they are located in the lower stories.

SUMMARY

Acceleration demands on non-structural components in low and high-rise ductile buildings are examined under severe and moderate intensity levels of shaking, specifically ultimate limit state (ULS) and operational or serviceability limit state (SLS) intensities. The findings, similar to those from prior research studies discussed in the 'Code Provisions' section of the paper, reinforce the view that amplification of floor spectral response is greatly affected by the structural behaviour, especially at frequencies close to the fundamental period of the building in such a way that: (i) there is considerable amplification of acceleration response when the building behaves linearly, and (ii) there is much less amplification when the building behaves nonlinearly. However, for short periods, invariably larger spectral amplification is observed for both ULS and SLS conditions than at longer periods.

Overall conclusions are:

1. Code provisions are adequate in ULS conditions for both short-period and long-period components in the low-rise and high-rise building categories considered in this study. However, in SLS conditions, the code provisions are (i) adequate for short-period components in high-rise buildings, (ii) inadequate for long-period components in upper stories of high-rise buildings, and (iii) marginally inadequate for short-period components in lower storeys and for long-period components in upper storeys of low-rise buildings.
2. The peak floor accelerations up the height of a building are much less than the increase suggested by the NZS provisions under both SLS and ULS shaking intensities for Wellington. It can be appreciated that this observation is supported by many other researchers (as cited in this paper). However, authors feel that further research, including both analytical investigations and recorded measurements, will be helpful to confirm this aspect.
3. The values for component amplification factors within NZS provisions are found to be marginal, especially for SLS shaking intensities. The amplification ratios (spectral response divided by peak floor accelerations) are often found to exceed the NZS provisions which specify a maximum value of 2 for component periods less than 0.75s.

4. Because of the counteracting effects of highly conservative values for the floor height coefficient and unconservative values for the component amplification factor compared to the analytical results, it appears that the code design envelope are generally adequate for the design of components with periods less than 0.75s under ULS conditions.
5. Under SLS shaking conditions, the NZ design envelope provides considerably lower design accelerations for non-structural components with periods close to the building period for high-rise buildings and sometimes in low-rise buildings.
6. Component amplification factors expressed using T_p/T_{B1} better represent the period range influenced by the 'resonance effects' so that peak amplification of floor acceleration can be incorporated, particularly in SLS conditions.
7. From the study limited to only a single low-rise building, it appears that a subduction slab seismic event could prove quite damaging for short-period components located in this type of building both in ULS and SLS conditions.
8. The findings of this study should apply to floor accelerations at the same normalized building height (floor height over total height) and normalized component period (component period divided by the building natural period), irrespective of the number of storeys in the building.

It appears that design criteria for non-structural components should account for the intensity of ground shaking and the corresponding structural dynamic behaviour (elastic/inelastic), so that the possible increase in acceleration demands under lesser intensity of shaking can explicitly be accounted for, avoiding damage to components and ensuring operational continuity of buildings.

The present study considered only ductile buildings and it showed that the demand could be higher during lesser intensity shaking. Further research on buildings that are likely to respond elastically, or nearly elastically, under suitable levels of earthquake intensity is to be undertaken, with the aim of confirming the extent of increase in loading demands on components.

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