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BASIN EDGE EFFECTS AND DAMPING: A STRUCTURAL ENGINEER'S VIEW

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ABSTRACT

The Kaikōura earthquake brought the concept of basin effects to the forefront of conversation about building in the Wellington CBD. Local exceedances of ULS design spectra were observed in many waterfront sites in the 1.5-2.5s period range. This, coupled with low yield levels and certain structural forms present in previous generations of building design, meant that significant damage occurred in many buildings around the Wellington waterfront.

A primary cause for the high spectral accelerations was the geological structure of the Wellington CBD. This paper will focus on the behaviour of generic buildings in response to these particular ground motions and suggest how lessons from this can inform the design of future buildings. It uses the Kaikōura Earthquake as the centre point for discussions about the relationship between building behaviour on soft soils and the effects on this of different forms of damping. More broadly, the aim is to help spark debate in the earthquake engineering community on the question: What sorts of structures should we be building on soft soil sites?

This paper has been written in the wake of a number of damaging earthquakes throughout New Zealand, and with the concurrent increase in sophistication and spread of tools for analysing the effects of the ground motions induced by these earthquakes. The genesis of the ideas presented herein was in analysis of many waterfront buildings following the Kaikoura earthquake, and the attempts, often in vain, to match modelled building behaviour- where small tweaks in assumptions could have a radical effect on results- with actual observed damage – where cracks may have been seen in concrete or in partitions, but assessment of actual plastic strains reached in steel bars or beams was basically conjecture.

This paper is broad in scope, therefore cannot possibly give each aspect the coverage of a series of papers which consider them in isolation and in detail. We nonetheless strongly believe that a holistic view of all topics is critical for design, and that the authors as 'front line' structural engineers are well positioned to present this. Sincere attempts have been made to justify our point of view with a strong basis in first principles, and backed by nonlinear time history analysis, or by reference to the work of others. We acknowledge that our beliefs are not shared by everyone and that some conclusions are provocative. It is neither the intent nor even the hope that we have the last word on this topic.

INTRODUCTION

Wellington Geology

The Wellington CBD is dominated by two geological basins. One is on what was the Te Aro Swamp, and consists primarily of swamp sediments and colluvium intersected by alluvial deposits of current and former streams. It extends from the waterfront north of Te Papa to about SH1 at Arthur Street, and from Victoria Street in the east to Cambridge Terrace in the west. It is up to around 150m deep. The latter bowl extends from around Molesworth Street in the west to the water, and the surficial layers primarily consist of 19th Century reclamations. It consists of quarried rock, domestic waste and other fill from Molesworth Street to approximately the Wellington Stadium, with hydraulic fill beyond, all overlaying natural marine deposits, with bedrock at up to 250m [1]. These basins, along with the known site periods for Geonet Strong Motion Stations can be seen in Figure 1.

These geological structures make the area vulnerable to basin effects. These effects fall into three main types. First is basin impedance- this is the magnification of seismic wave amplitude resulting from the transition from a hard to a soft soil layer, particularly at the site periods. Secondly, basin edge effects are the constructive interference between S waves and the surface waves they generate when they meet the discontinuity at the basin edge. The third effect, basin reverberation, is caused when seismic waves are 'trapped' within the basin due to the structure of the edge, and amplitude and duration of motion is increased.

One or all of these effects have been shown to be a cause of significant damage in previous earthquakes - namely at Kobe in 1995 [2], Seattle in 2001 [3], and at the Santa Monica Basin in the 1994 Northridge earthquake [4]. Damage in Kobe was primarily to long-period timber houses and reinforced concrete frame buildings in the 3-5 storey range, with wall buildings performing significantly better [5]. Damage in the Santa Monica region was to steel moment frame buildings [6] and poorly detailed reinforced concrete buildings, particularly frame and tilt-panel construction, with shear wall buildings again typically performing better [7].

Basin edge effects in Wellington were the subject of some study in the early 2000s [8], but this work did not appear to influence discussions nor policy, at least as far as the structural engineering community is concerned. This paper predicted amplifications in the Te Aro basin in the 0.7-1.3s range. Basin

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effects were also hypothesised for the Lower Hutt region, however these do not appear to have resulted in amplifications above design ULS levels in the Kaikōura event.

Figure 1: Basic Wellington geology with geological basins shown red (based on soil C/D transition zone [1]); SGM stations are shown with site periods where known.

The Tools of a Structural Engineer

The 5% damped spectrum is the primary way knowledge about how ground motions work is converted into information about how buildings should be built. Seismologists use a wealth of knowledge about earthquakes and ground behaviour to produce a plot of the expected maximum accelerations of buildings with various ratios of mass to stiffness. Engineers then use this information to apply various design tools to account for how buildings and their constituent parts behave. What happens during actual ground motions is a complex interaction between the topics of study of each, with peculiarities and nonlinearities which are not present in generic analyses.

What a spectral response represents in structural engineering terms is a snapshot of a building's displacement at the instant during ground motion when it is at its maximum, thus reducing a complex dynamic problem to one which can be solved with static equations. What is obscured is how it got there, through many low-amplitude cycles or a few high-amplitude cycles, and what will happen after, whether it be many continued cycles of similar amplitude or an abrupt end to movement.

To reduce building costs, and to control ground motion movements, buildings are typically designed for designated elements to yield and enter the inelastic range. For example, a concrete moment frame building has specially detailed 'plastic hinges' in the beams. When the inertial force induced by ground shaking reaches a certain threshold, the longitudinal reinforcing steel in these plastic hinges will yield and elongate, absorbing energy as they do. This results in ductility and so-called 'hysteretic' damping. This reduces forces and sometimes displacements by absorbing ground motion energy through both plastic deformation and so-called 'period shift'. In general, designated yielding regions have specific design criteria, with stricter limits on deformation capacity and stability requirements.

The basic analysis tools a structural engineer uses to account for inelastic behaviour - the equal displacement principle in force-based-design, and equivalent viscous damping and effective stiffness-based period shift in displacement-based design – are intended to be a simple and generic approach to many different types of ground motions. Their applicability to specific ground motions may be questionable.

The equal displacement 'principle' is based on observations originally made in the 1960s based on results of inelastic response history analysis of three earthquake induced ground motions [9]. It states that an inelastic system will have the same displacement as an elastic system of the same natural period. It has no known basis in first principles, and it does not appear to hold for many unique ground motion phenomena, nor to large magnitude earthquake induced ground motions in general [10]. It has nonetheless persisted in design codes, due to its rough applicability and its simplicity. Using elementary algebra, one can show that- if the observation holds- one can design for inelasticity by dividing the elastic force by ductility level assumed. This is a remarkable coincidence, the simplicity of which frequently obscures both the heuristic nature of the observation itself, and the complex behaviour which underpins it. As Paulay and Priestly [11] eloquently state, "This observation is sometimes referred to as the equal-displacement principle, although it does not enjoy the theoretical support or general applicability to warrant being called a principle."

Rather than rely on heuristics, this paper makes frequent use of constant ductility spectra. Essentially this process performs nonlinear time history analysis for a range of periods for elastic-perfectly plastic systems, and iterates the system strengths until a given ductility level is reached- for more details of the details and theory behind this, the reader is referred to the literature [12]**.** This process was conducted using the SeismoSignal program. A similar analysis was undertaken by Carr for selected records for the Christchurch earthquake sequence [13].

THE KAIKŌURA EARTHQUAKE

The 2016 Kaikōura Earthquake was a large magnitude (M_w = 7.8) earthquake which propagated in a complex path along many faults on the east coast of the North of the South Island. Significant damage has been identified in and around the Wellington CBD basins, as shown in Brunsdon et al in Figure 2. This paper will not cover the tectonic setting, rupture, and ground performance - the details of which have been much explained by others [14-17] - except in so far as they relate directly to observed building behaviour.

At the point the ground motions from the Kaikōura earthquake hit Wellington, both the Peak Ground Accelerations (PGA) and spectral accelerations were below ULS levels for accelerometers on rock [11]. For the soft soils in the aforementioned basins, the PGAs were also lower than ULS levels - typically around half. Some significant exceedances were observed in the spectral accelerations for many waterfront sites, particularly around the site periods of the general area; the amplifications of soft soil sites relative to rock sites were significantly higher than the amplifications in NZS 1170.5, as were the amplifications of many spectral accelerations of soft soil sites relative to the PGA.

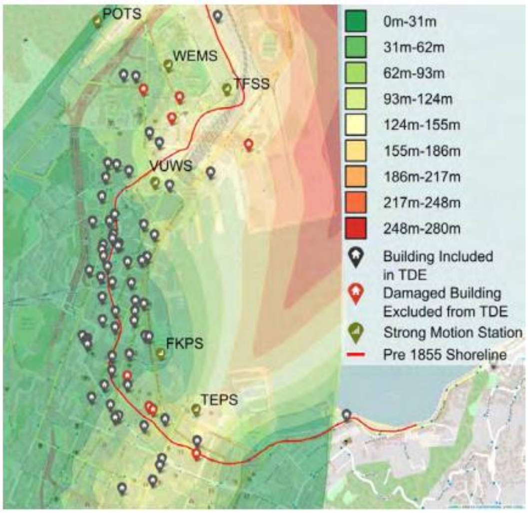

Figure 2: Distribution of damage buildings overlaid over distance to bedrock. From Brunsdon et al [18].

The magnitude, distance and duration of the ground motions contributed to the high spectral accelerations for long periods–high frequency waves are attenuated more quickly (with time and distance), than long period waves. The long period, long duration waves were then amplified and extended by effects in and around the Wellington basins. Figure 3 shows the time history plot of the ground displacement measured at the TEPS station against the behaviour of an elastic T = 1.5s natural period single degree of freedom system (SDOFS) with 5% viscous damping. The maximum displacement reached for the structure is built up over several cycles, with a maximum ground displacement of around a third of the peak displacement the structure reaches. The reason for the high spectral displacement is a dynamic amplification commonly associated with resonance.

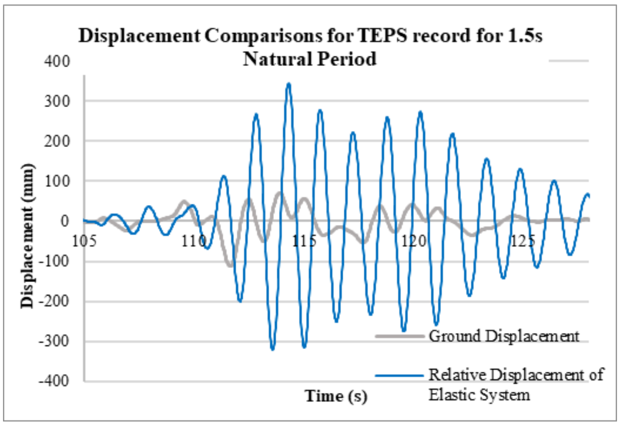

Figure 3: Response to TEPS record of a Tn = 1.5s SDOF system.

Resonance is the build-up in response of a system when the natural period of the system matches the period of the forcing function. It occurs in systems which can easily store and transform energy from one form to another. In the case of buildings this is between elastic potential energy stored in the structural system and kinetic energy. When energy is not stored, but is dissipated as heat, the build-up of energy is reduced, and the degree that resonance can increase the response of a system relative to the amplitude of the forcing function is capped. This can be seen in the displacement plot in Figure 4 below. This shows SDOFS with a T = 1.5s initial period with elastic and ductile responses which are subjected to the portion of the TEPS Kaikōura record that gives the maximum spectral response. Displacement is chosen as it allows for more meaningful comparisons between elastic and ductile systems. While not exactly in phase with the ground movement, the action of the ground motion (in grey) below builds up displacement in the elastic system (in blue) over several cycles. The ductile system (in red) can be observed to yield at around the T = 111s mark, which puts it almost exactly out of phase with the ground motion, and the response actually decays over subsequent cycles. The actual period elongation is slight, (equivalent to an increase from 1.5s to 1.7s for the first quarter cycle) but is enough to reduce the displacement significantly. This behavior can be seen more generally in Figure 5.

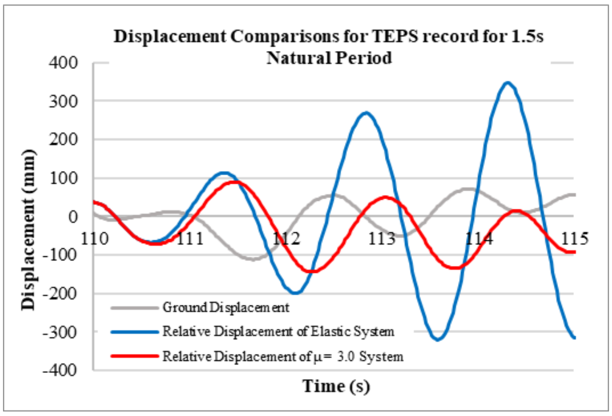

Figure 4: Response to TEPS record of a Tn = 1.5s SDOF system against a Tn = 1.5s ductility of 3 SDOF system.

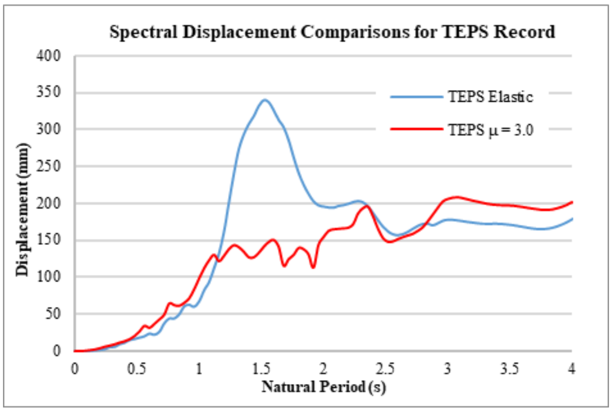

Figure 5: Maximum spectra displacement plot for elastic and ductile systems for TEPS record.

This effect can also be seen in spectral acceleration plots of the Kaikōura earthquake ground motions response compared with spectral accelerations predicted in NZS1170.5 (Figure 6). As noted, at the 5% viscous damping ratio generally assumed, the acceleration is higher for some periods. When the ductility is increased to 3, a typical number used in design, the acceleration drops below the NZS1170.5 levels for that ductility level for virtual all periods considered. This is because hysteretic damping is particular effective for ground motions at these sites. Similar reduction of ‘bumps’ in spectra has been observed in Carr for the Christchurch earthquake ground motions [13] and by Krawinkler for the Loma Prieta Earthquake motions [19].

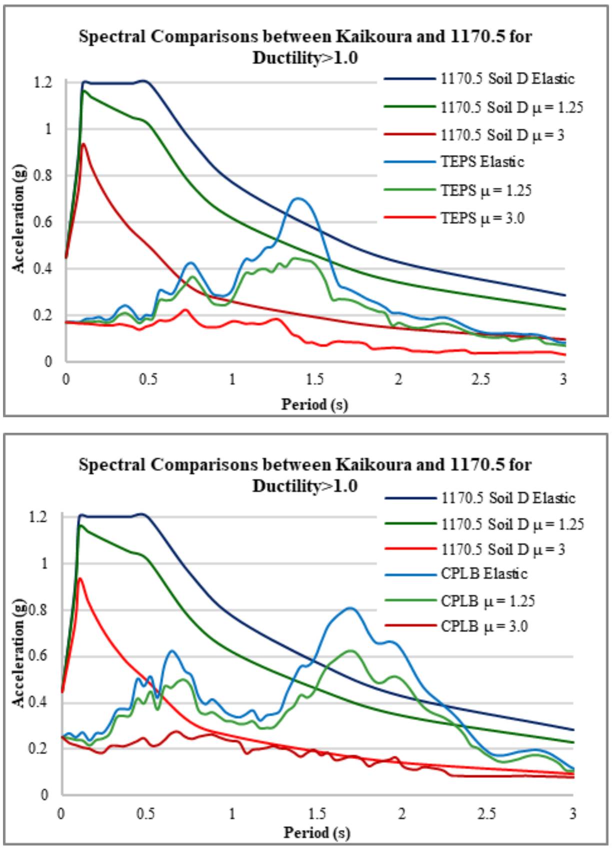

Figure 6: Maximum spectral acceleration plots of elastic and ductile systems compared with NZS1170.5.

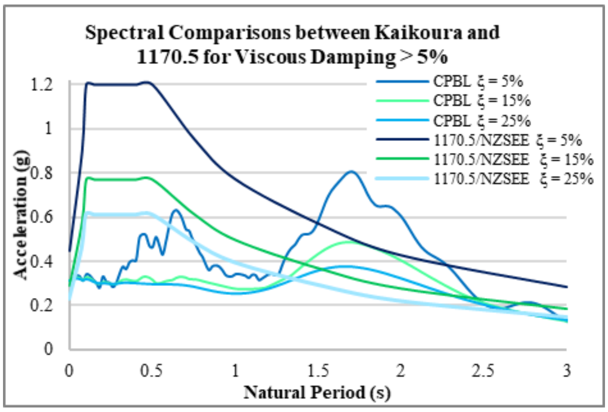

Figure 7: Maximum spectral acceleration plots of elastic systems with varying levels of viscous damping compared with NZS1170.5/NZSEE.

For viscous damping, a much less commonly used form of damping in structural design than hysteretic (ductile) damping, a similar but less pronounced effect can be seen in Figure 7. The primary reason for the reduced effect is that the dominant ground motions are generally at fairly discrete natural periods. The advantage of hysteretic damping in this case is that at a certain acceleration threshold, the stiffness drops significantly and the system starts to dissipate energy rather than storing it, meaning it will not snap back in phase when the earthquake ground motion reverses. As the structure displaces, the period will elongate, and the structure will begin to move out of phase with the ground motion forcing function. Viscous damping around the 5-30% range has only a small effect on the natural period of a structure, therefore is not as effective at reducing response.

This phenomenon can be seen for the majority of soft soil waterfront sites in Wellington in Figures 8 and 9. Plotted in these figures are the hysteretic (μ=3) and viscous (ξ =20%) damping reduction factors for soft soil sites in Kaikōura, compared with a suite of records chosen by GNS for a Soil D waterfront site in Wellington in 2016. At the natural periods of greatest spectral amplification, the effects of damping are simultaneously at their greatest. Again, this effect is most pronounced for hysteretic damping in the Kaikōura ground motions, where the reduction factors are as high as 8, over twice the kμ reduction factors generally used in design in this period range (kμ = μ = 3).

For the Soil D records, while much scatter exists in the data – with predictions between 0.5 to 2 times any individual record, reductions based on equal-energy theory fit the average of the data well up to 0.7s, with the equal displacement principle fitting moderately accurately beyond this; the NZS1170.5 kμ reduction factors, are generally within 20%.

Again, for the Kaikōura records for the Wellington waterfront, the viscous damping reductions are higher than normal in the 1.25-2.0s range – up to nearly 2.5 times mean reduction at 1.5s compared with the 1.5-1.8 times reduction for both the Soil D records, and for the standard viscous damping reduction factors in the NZSEE Assessment Guidelines below (1), and the similarly formed factor from Eurocode 8 (2), with the Eurocode formula matching the Soil D records more accurately. Note that in the 0-0.5s period range, standard formulas are non-conservative as damping is not effective in the short period range. This is a well understood phenomena [20], and is recognised in Eurocode 8, with smaller reductions for short periods.

$$K_\xi = [7/(2 + \xi_{sys})]^{0.5} = [7/(2 + 20)]^{0.5} = 1/1.77 \qquad (1)$$

$$\eta = [10/(5 + \xi_{sys})]^{0.5} = [10/(5 + 20)]^{0.5} = 1/1.58 \qquad (2)$$

where, K_ξ / η = the inverse of the reduction factor; and ξ_{sys} = the system damping.

Interestingly, the hysteretic damping reduction factors for the Kaikōura ground motion records are less than average for the Soil D Records at periods just under that of the peak spectral accelerations. This is partially due to the dip in amplification at these periods, and partially due to the period elongation previously mentioned, which in this case may move the structure in phase with the dominant ground motion frequency. This can be observed in Figure 10 below. Initial behaviour is similar to that of the 1.5s system mentioned previously, but a single large ground lurch of period ~2.0s extends the T=0.7s period to nearly 2.0s (for the first quarter cycle). After the system yields, further deformation is not met with a corresponding increase in force resistance, resulting in significantly increased relative displacement between the ground and the structure as the structure is ‘dragged’ in phase, to use a simplistic analogy.

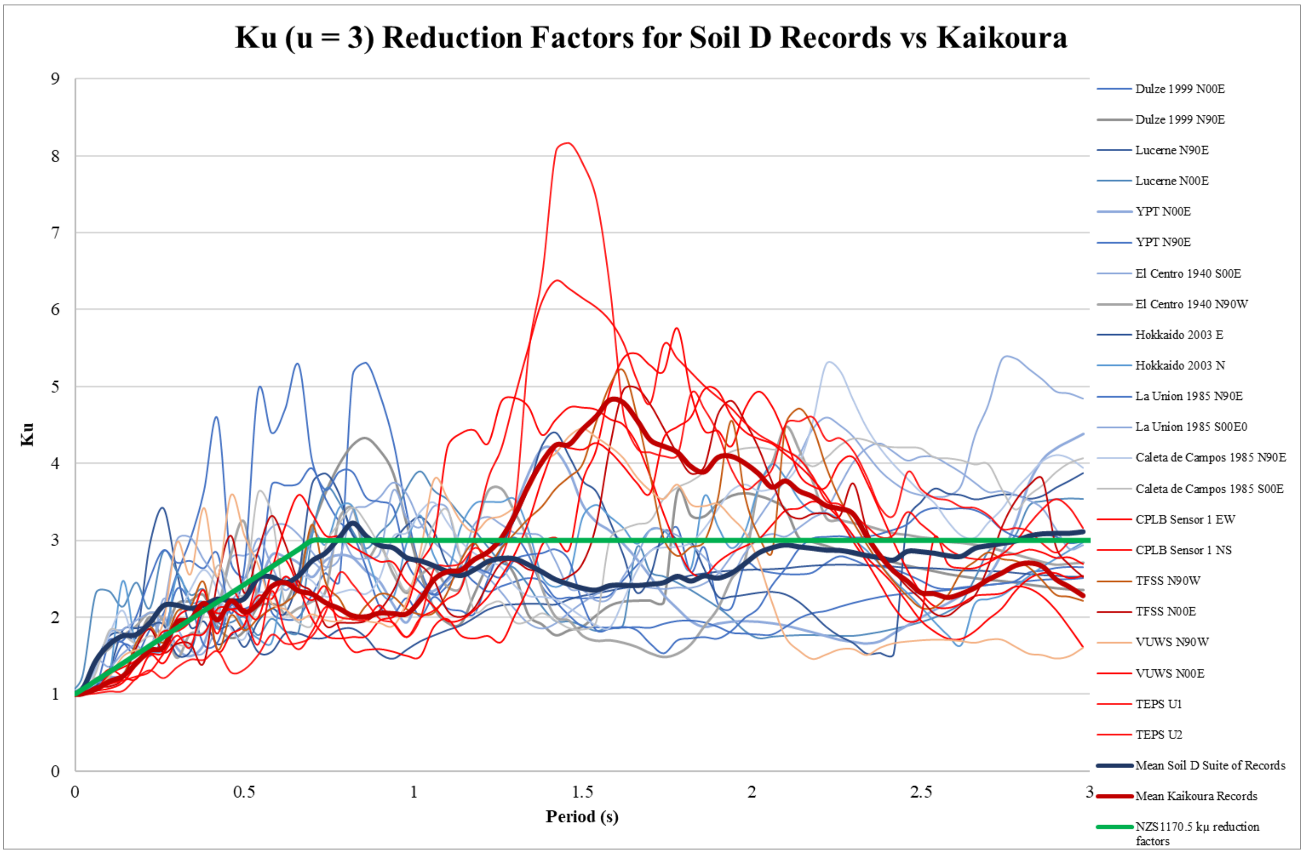

Figure 8: Hysteretic damping reduction factors for Kaikōura records against records chosen for a waterfront site in Wellington.

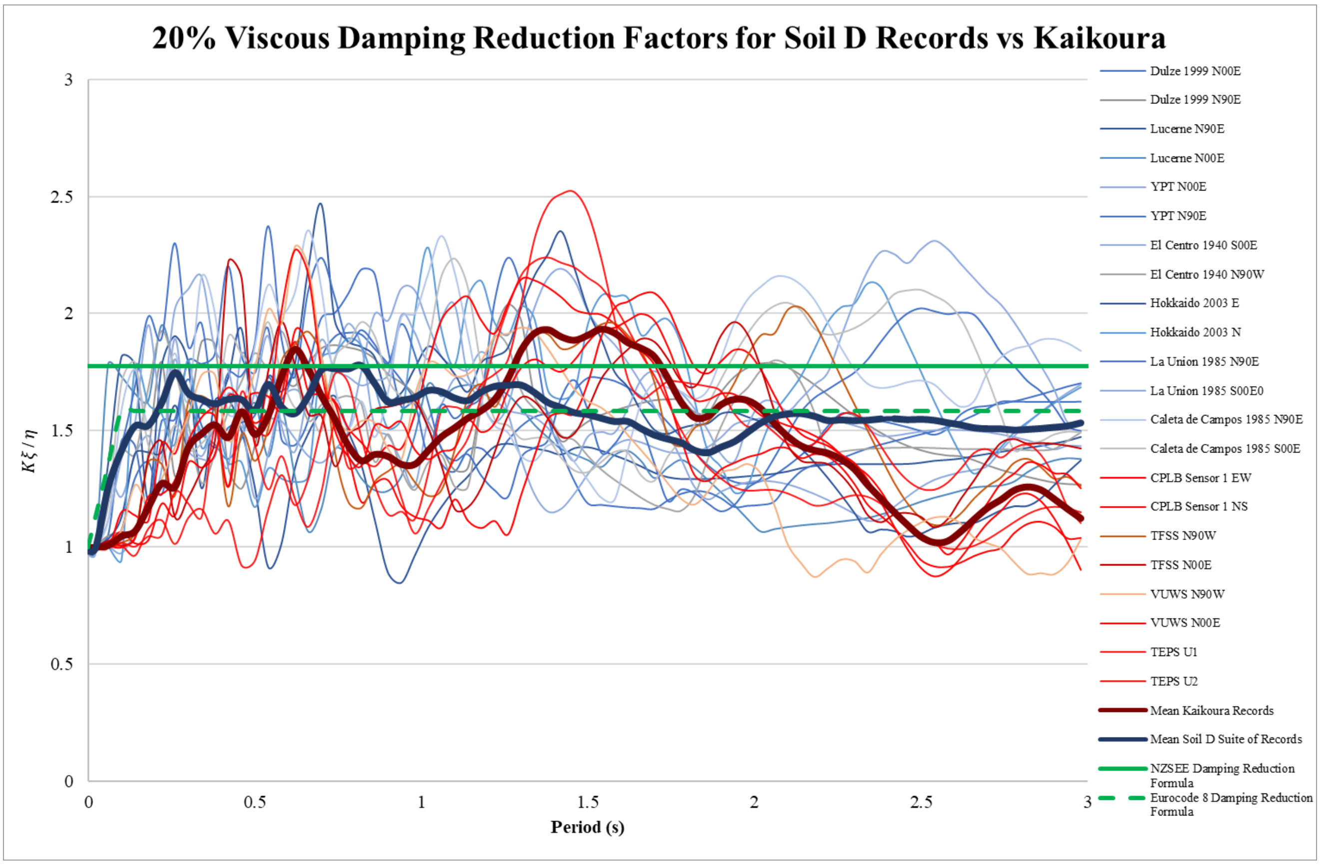

Figure 9: Viscous damping reduction factors for Kaikōura records against records chosen for a waterfront site in Wellington.

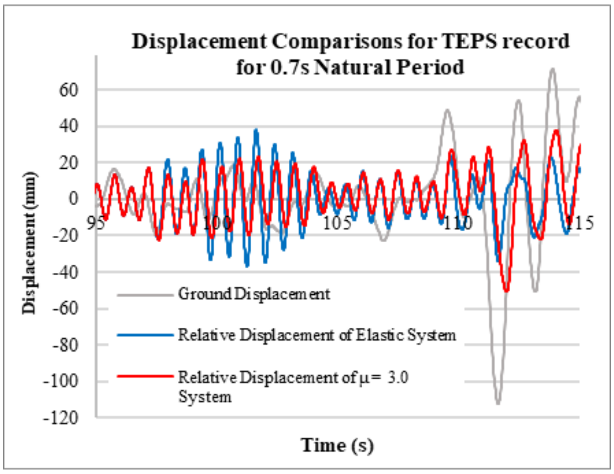

Figure 10: Response to TEPS record of a Tn = 0.7s elastic SDOF system against a Tn = 0.7s ductility of 3 SDOF system.

It should be noted, however, that there is still a net decrease in response in general. This is because a) ground motion energy is still damped out by the hysteresis and b) the period elongation will move the building in phase with the ground motion for the cycle out to the maximum response, but the building will not store much energy, so it will not snap back in phase with the ground motion and with the full elastic stored energy of a building that has not yielded. The period of the building is also changing throughout the response, limiting build-up of dynamic amplification.

The equal displacement principle underpins the kμ reduction factors for periods greater than 0.7s for force-based design in NZS1170.5. In Figure 11, it can be seen that this behaviour holds approximately for the 2.5-4.0 second range for ductility =3. At the period of the 'bump' in the spectra, the reduction in displacement due to ductile damping is close to 2 times, meaning this form of damping is significantly more effective than that assumed in NZS1170.5.

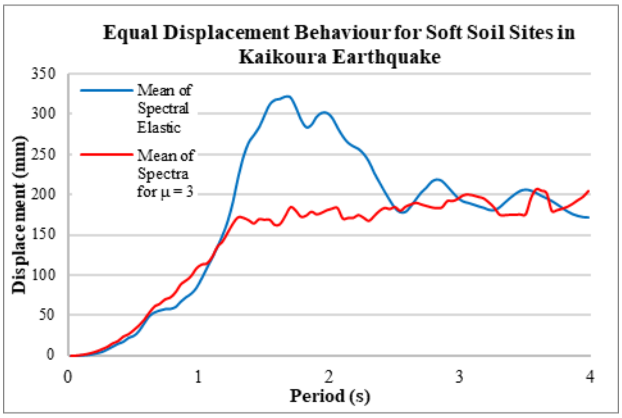

Figure 121: Maximum spectral displacement plot for elastic and ductile systems for average of Kaikōura soft soil records.

A most pertinent question to ask at this point is why, if the reductions are so significant, was so much damage observed? To help answer this, we will look at Acceleration/ Displacement Response Spectrum (ADRS) plots in Figure 12. The relationship between the different damping levels in the hysteretic and viscous damper ADRS plots has an interesting physical interpretation. In these plots, the concept of building period (whether it be the natural or the effective period) is obscured, so the period shift that happens when a structure yields and loses stiffness is decoupled from the energy absorption part of hysteretic damping – the energy absorption aspect is shown more clearly.

The difference in magnitude between the elastic spectra and the hysteretic spectra approximately represents the amount of energy absorbed by the system. As can be clearly observed from this plot, the difference is very pronounced, indicating a very high amount of energy absorption. This is a problem with systems which are not well set up for it – concrete moment frame buildings with inadequately seated precast flooring in particular; buildings with difficult-to-replace damping mechanisms or with degrading strength and stability in general.

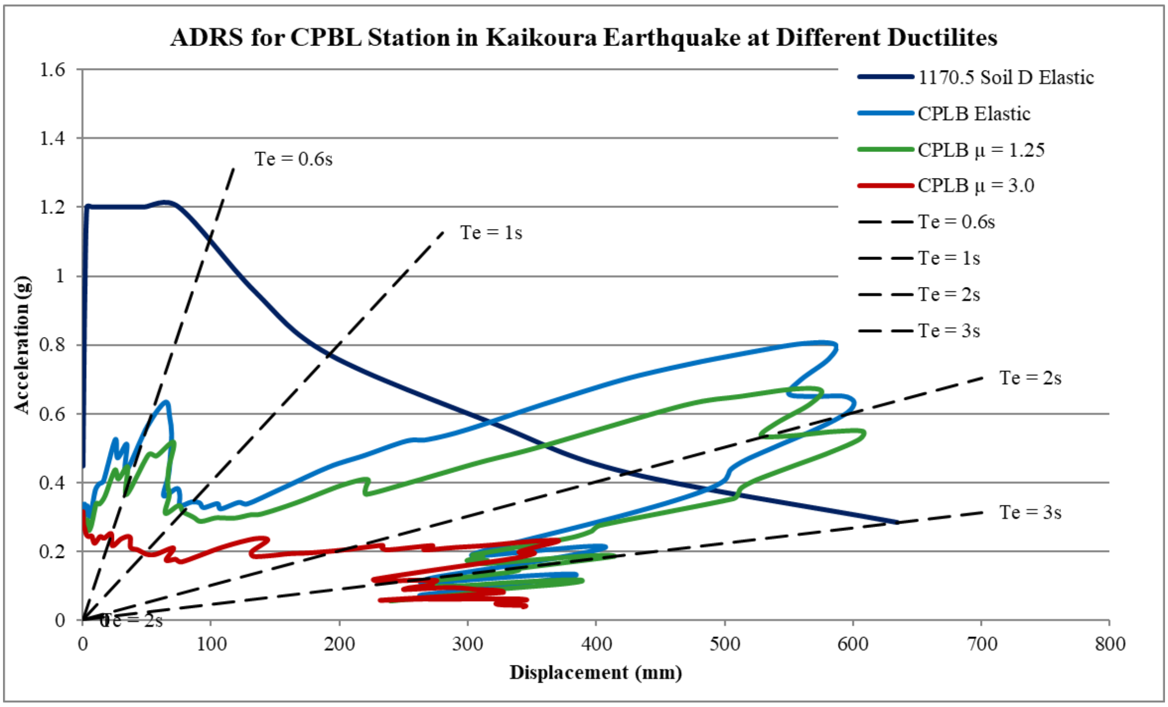

Figure 112: ADRS plot for CPLB record at varying ductility levels.

Such buildings will perform worse than a building cycling in the elastic range. For buildings with good energy dissipation, the opposite is true. Lead rubber base isolators, for example, can cycle almost indefinitely at displacements below their ultimate design displacement, with only moderate and stable reductions in strength due to lead core heating. A low-damage ductile system will likely also have a much more moderate and predictable response.

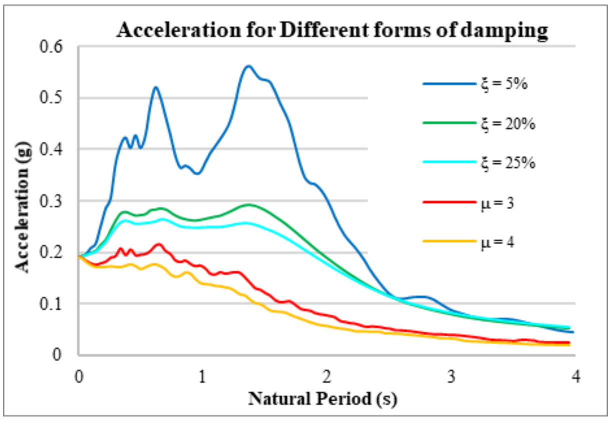

Figure 13: Average spectral accelerations for Kaikōura soft soil records at different levels of damping.

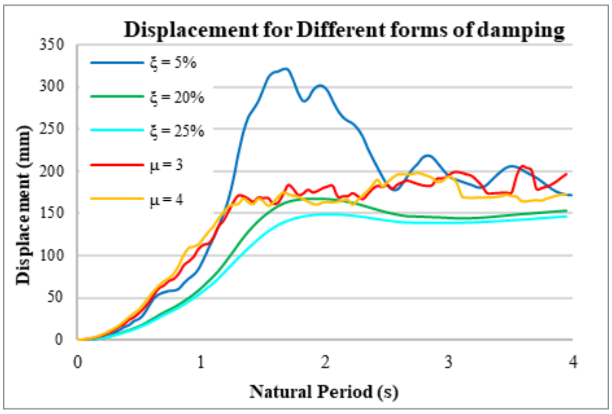

Figure 14: Average spectral displacements for Kaikōura soft soil records at different levels of damping.

The analysis above appears to suggest that hysteretic damping is relatively more effective than viscous damping for these particular ground motions at these locations. The truth is that the behaviour is a little more subtle. Figure 13 and Figure 14 show acceleration and displacement behaviour for the mean of the Kaikōura records, assuming different levels of damping. Firstly, it is apparent that the equal displacement assumption is a poor predictor of maximum response in the 1-2.5s range, though it holds fairly well once the 'bump' has been eroded and beyond 2.5s. It is also clear that hysteretic damping is much more powerful at reducing force demands than viscous damping. Thirdly, and broadly speaking, hysteretic damping has limited ability to reduce building drifts compared with a system that does not have to lose stiffness to absorb energy (beyond reduction of the 'bump'). It is also worth noting that truly viscous damping will be the same across all limit states, so will be effective in small as well as large events, whereas hysteretic damping (the level of which is calibrated with strength to match a design-level event) will vary across events and limit states.

An additional observed behaviour is the effect of post-yield stiffness/restoring forces. Figure 15 shows an interesting phenomenon which is difficult to see in spectral plots: the displacement time history response of SDOFS with high ductility (μ =6) for the CPLB record. As can be seen, a system with no restoring force can undergo cumulative displacement in response to ground motions as it has no inherent ability to return to its centre point. A general smoothing and reduction in maximum displacements can be seen in Figure 16. P-delta actions have significant interaction with post elastic stiffness. As a building displaces, it loses stiffness due to the overturning moment induced by the gravity and building displacement. This is generally much lower than the elastic stiffness, so it does not become an issue for elastic buildings at code-level drifts. When a system yields and has no inherent restoring force, the post elastic yield slope will be negative, which tends to drastically increase displacements [17]. Having a positive restoring force will offset this effect. P-delta effects are not included in the simulations used in this paper.

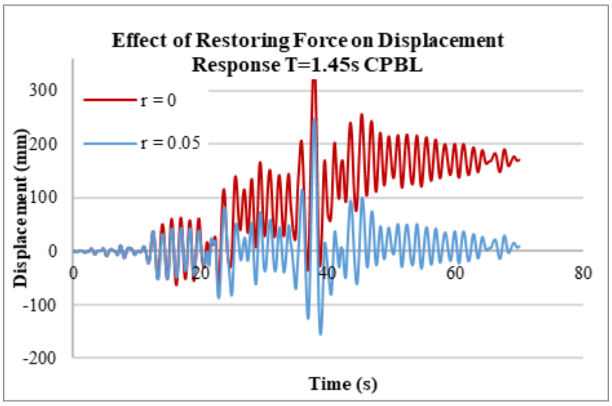

Figure 15: Displacement response of T=1.45 ductile and elastic systems r=0.05g restoring force.

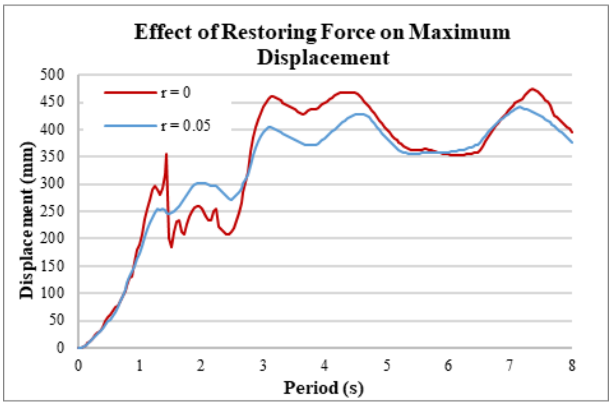

Figure 16: Displacement response of u = 6 systems at varying restoring force.

DESIGNING FOR FUTURE GROUND MOTIONS ON SOFT SOIL SITES

Serviceability Limit State

The Kaikōura earthquake highlighted the potential inadequacy of buildings with low strengths and damage-based hysteretic systems. That many buildings had to be completely demolished due to the performance of the seismic structural system is a serious economic issue.

The converse of the aforementioned point about damping reductions is that when damping is less than the 5% assumed, basin effects will more pronounced. Analysis of buildings elsewhere in the world [21] suggests that - certainly in the elastic range - buildings do not universally have the 5% damping generally assumed. Consider also that many buildings are medium-rise, 1-2 storey structures whose response will be highly amplified and therefore subject to significant damage in moderate ground motions.

This raises questions about the Serviceability Limit State (SLS) criteria used in New Zealand - which is low compared with other countries [22] - and its ability to meet the Building Code's fundamental requirements.

Additionally, for buildings with lower inherent damping than 5% - for example tall buildings or 'bare'/simple structures- an increased design spectrum should be used where basin effects are prevalent. This is important even at the current 1/25 (SLS1) annual probability of exceedance level and would likely necessitate the development of basin specific damping 'reduction' factors.

Ultimate Limit State

It has been noted that the basin effects can be expected to be repeatable - if perhaps not identical - across virtually all future earthquakes [14, 23]. What can then be expected from a rupture of higher magnitude, longer and closer earthquake? Below in Figure 17, we see 4x the CPLB with record spectra for Hikurangi on Rock [24] overlaid for comparison, and an ADRS plot for Wellington Soil E for base isolation design (Figure 18). The latter gives similar amplifications of soft soil sites relative to rock to those observed in Kaikōura. Any comparisons between The Hikurangi spectra and the scaled CPLB record are crude- as will be mentioned later soil nonlinearity will tend to change the shape of spectra from large magnitude earthquake ground motions. It does, however, use a similar scaling philosophy to the current New Zealand loadings code, and the 'absurd' 4x scaling was based on the approximate maximum displacement capacity for Building A below.

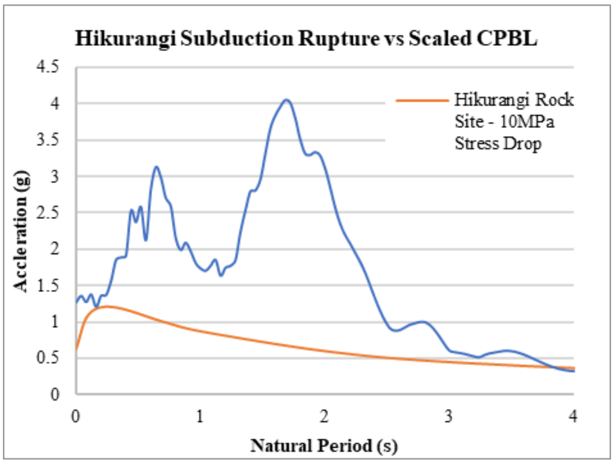

Figure 17: Hikurangi spectra with 4x CPLB record overlaid.

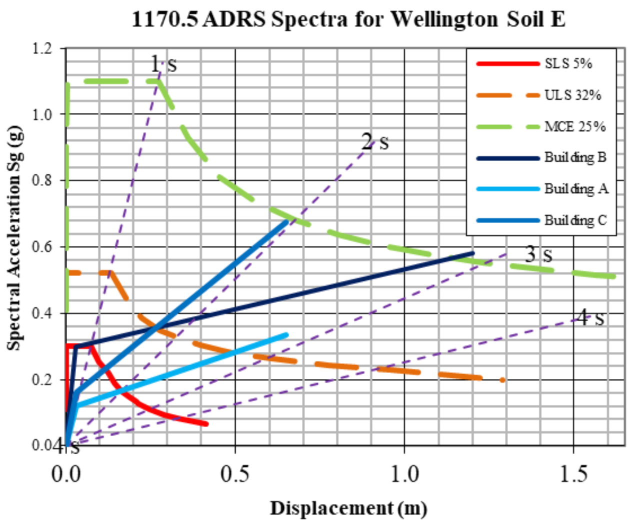

Figure 18: ADRS Spectra for 3 base isolator (BI) designs.

The first thing one should countenance with design parameters this extreme is: should we be designing buildings in areas this vulnerable? To help answer this question we will begin by examining the behaviour of sample Importance Level 2 Base Isolated Buildings to current code (including Soil C-D interpolation), and two designs assuming Site Soil Class E and 25% damping at MCE. One of the Soil E designs is based on a typical Lead Rubber Bearing Displacement (650mm), and the other (1200mm) is based on either an extreme LRB design, or a triple friction pendulum system with added damping. Backbone curves for these buildings are plotted in the graph above.

Table 1 below shows displacement and acceleration response for each of these base isolation schemes (assuming a yielding SDOFS) at 1, 2, 3 and 4 times the CPLB Kaikōura record. The nonlinear time history analysis was conducted with ETABS software. Despite the much more significant design levels for the Soil E design buildings, they do not necessarily have better performance than the C/D interpolation buildings when subject to the CPLB record. Displacements at higher levels of loading are not reduced relative to the C/D building, and accelerations are higher at low levels, which will increase non-structural damage. Generally, the designs necessitated by the increased spectra produce buildings with responses closer to the 1.5-2.5s peaks. It also highlights the fact that increasing building strength is not in itself a guarantor of safer buildings.

Table 1: Response of BI buildings to CPLB at varying scaling Factors (SF); listed by soil type design and rattle space size.

	Building A (Soil C/D 650mm)		**Building B (Soil E 650mm)**		**Building C (Soil E 1.2m)**	
CPLB SF	δ (mm)	Acc (g)	δ (mm)	Acc (g)	δ (mm)	Acc (g)
1	190	0.17	63	0.30	213	0.19
2	353	0.22	193	0.42	387	0.23
3	504	0.28	525	0.66	578	0.28
4	694	0.33	773	0.88	772	0.32

An explanation for the latter phenomenon is that due to its high level of damping, the building has reached steady state response. The formulas below are based on simple sinusoidal loading [20], and with linear viscous damping, though consideration of them gives insight into response for more complex loading.

$$u_0 = \frac{(u_{st})_0}{2\xi} \tag{3}$$

$$\frac{|u_j|}{u_0} = 1 - e^{-2\pi\xi j} \tag{4}$$

where u_0 = the maximum response;
u_{st} = the amplitude of the forcing function and
u_j = the amplitude in the jth cycle (for the plot below u_j is chosen as 90% of the maximum response).

At low levels of damping, the degree to which the ground acceleration can be amplified with continued cycling is very high, and the response will build with every cycle the building is subject to. When the damping is high, the amount the response can build is considerably less, and after just a few cycles, the amplitude of the cycling will not continue to rise (Figure 19). Therefore it does not particularly matter whether a system with 25% viscous damping does 2 cycles or 20: the maximum response will always be the same. A similar phenomenon will occur for ductile damping, though this is difficult to represent analytically.

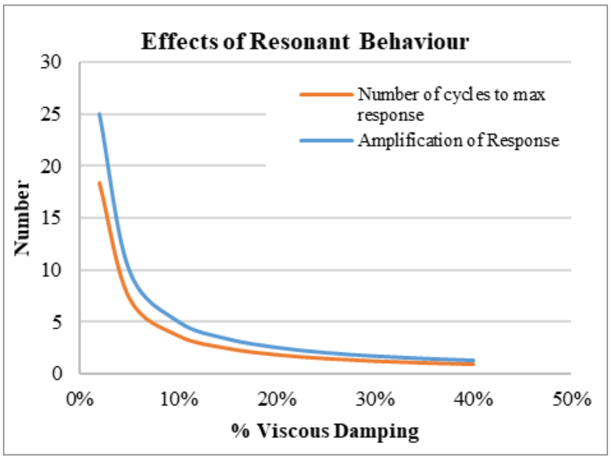

Figure 19: Effects of damping on resonant behaviour of SDOF system under sinusoidal loading.

A similar phenomenon may also occur in the amplification of soil response relative to rock response. As accelerations increase, soil nonlinearity becomes more likely and response appears to be capped at the dominant period and increased at periods beyond [25,26] (though that is outside the scope of this paper).

The analysis presented in this paper is for simple single degree of freedom systems. Actual buildings have many modes, and more complex force displacement loops. The analysis above applies to damping reductions of the first mode only and for elastic-perfectly plastic systems; one can expect some inaccuracies when the conclusions drawn are applied in a more general sense. Damping of higher modes is usually less than for the fundamental mode, which would mean they do not experience the same reductions. This would mean they contribute disproportionally to building drifts when basin effects are present and that NZS1170.5 drift modification factors may be non-conservative for buildings subject to these effects.

The elastic-perfectly plastic hysteresis rule is a reasonable approximation of behavior of steel buildings, though may be inaccurate for concrete buildings. These generally modelled with the Takeda rule, which generally has smaller hysteresis loops. Due to their degrading stiffness, concrete buildings may have greater displacements and accelerations than the previous analysis shows. Further study of multi-modal systems with different hysteresis rules is required to confirm these hypotheses.

Potential Changes to Design Tools

It is the opinion of the authors that more complexity in design spectra is not warranted, unless this is accompanied by additional complexity of tools for dealing with the various phenomena which produce peaks and troughs in the 5% spectra. Because the amplification of response at certain building periods due to basin edge effects is due to the repeated cycling at a consistent ground acceleration, a sensible approach would be to allow higher reduction factors for reliable damping systems.

In displacement-based design hysteretic damping is modelled with equivalent linear methods. Correction factors are typically applied to area-based or Jacobsen-equivalent viscous damping. One reason cited for this is that the Jacobsen method assumes a resonant forcing function, which is not generally the case [27]. A logical approach for buildings on sites whose response is more resonance based would be to have greater reduction factors. This rule may be linked to the spectral ratio, i.e. the ratio of the acceleration at any point in the spectra to the peak ground acceleration, as the higher this ratio is, the more the response is due to resonant-like behaviour rather than amplitude of the forcing function.

Alternatively, modified damping reduction formulas with a larger exponent (hence higher reductions) could be utilised. The greater effectiveness of damping due to more repeated cycling is the opposite of the more well-known effect of forward directivity ground motions, where a single large pulse of ground motion will reduce the effect of damping, relative to more general formulas [28]. For forward directivity events, a large majority of the ground motion energy comes in a single large pulse, resulting in fewer cycles than is necessary for damping to be effective, therefore a smaller exponent has been proposed. More generally, this exponent could be related to the number of effective cycles a building is expected to undergo.

The literature is split on the degree to which number of cycles matters in relationship to the maximum cycle for buildings in general [29]**.** On one hand, P-delta overturning becomes an issue at certain drifts; a brittle member will fail abruptly at a certain force; and a crack will initiate at a certain strain in a ductile member. On the other hand, beam elongation has historically assumed to be highly dependent on number of cycles (though recent testing at the UoA suggests however that again, the maximum cycle may be the dominant input), and the propagation of a crack in a ductile member is influenced by number of inelastic cycles when the strain is high enough [30]. Therefore, it does make some sense to incentivise low-damage buildings through consideration of cycles - for example make the S_p factor driven by the ability of a system to handle repeated cycling.

This paper has demonstrated that hysteretic damping is more effective than viscous damping for reducing the amplification of acceleration witnessed in Wellington waterfront sites in the Kaikōura earthquake induced ground motions. This is primarily due to the 'period shift' that occurs in hysteretic systems. Some low damage systems – such as those with fluid viscous dampers- will not experience such dramatic reductions. This suggests that rather than offering greater reductions to fatigue resistant buildings, penalties should be applied to those building forms which are known to degrade.

Greater attention to the relationship between building natural period and the dominant ground period would also go some way to addressing the issues highlighted above. This may be in basic design decisions, where one may choose a stiffer or more flexible structural form so the periods do not match. This may be verified with appropriate selection of time history records, where inclusion of records with similar site response would more accurately capture this effect.

It is alluded to, though not proven, in this paper that the relationship between soil response and damping may hold generally for all soft soil sites. This has some support in the literature [31], and the formulas therein appear to match the large hysteretic damping reductions seen in the Kaikōura earthquake induced motions around the ground period.

A potential design-tool solution for the difference in ductility effectiveness between different soil types is the inclusion of direct ductile spectra in building codes. Significant work has been done to relate earthquake magnitude, rupture distance, and soil type directly into ground motion prediction models and spectra generation [32,33]. This approach would have several advantages: it would result in more accurate predictions of building behavior, and incentivise selection of ductility levels more directly appropriate for a given hazard. It is also significantly simpler to read a design coefficient off a graph, rather than follow a complex process which is not always understood.

CONCLUSIONS

Science in the construction industry should be the handmaiden to good building design. Focusing on ensuring each element of the code matches reality and adjusting them in a piecemeal fashion does not necessarily result in better performing buildings. The Kaikōura earthquake has highlighted the importance of reliable damping systems. It has shown that elastic and high-damage buildings will be vulnerable to future ground motions and has demonstrated the superiority of low-damage technologies.

The results of this paper strongly suggest that engineers should be cautious in drawing definite conclusions from 5% viscous damped spectra- both for spectra derived from individual ground motions, and for design spectra as the effects of ductility are different for different ground motions and different in various parts of the spectra.

We strongly suggested that further research is undertaken towards development of robust inelastic displacement spectra for eventual inclusion in design codes.

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