Method for the explicit consideration of ground motion duration in NZS 1170.5

V. Bhanu,
University of Canterbury, Christchurch, New Zealand
Beca Limited, Wellington, New Zealand

R. Chandramohan & T.J. Sullivan
University of Canterbury, Christchurch, New Zealand

ABSTRACT

Recent studies have demonstrated that long duration earthquake ground motions can reduce structural deformation capacity and increase collapse risk. This study introduces a method to explicitly account for such effects while designing new buildings in New Zealand. An equation is presented to adjust the design drift limit prescribed in the New Zealand standard NZS 1170.5, based on the target duration of anticipated ground motions at the site. The proposed method is used to derive designs corresponding to three duration targets for a 4-storey case-study steel moment frame building located on a site in Nelson. Hazard-consistent collapse risk assessment results indicate that buildings designed for lower drift limits have a lower mean annual frequency of collapse. The application of the proposed method is found to reduce the variation in the collapse risk of buildings designed for different duration targets, compared to the existing approach. Given that several sites in New Zealand are exposed to large-magnitude crustal and subduction earthquakes and therefore, long duration ground motions, the proposed approach can provide a better uniform-risk based design across different sites.

1 INTRODUCTION

Although modern code-specified response spectra, such as in the New Zealand seismic design standard NZS 1170.5 (Standards New Zealand, 2004), provide a fairly accurate estimation of the mean intensity and frequency content of the anticipated ground motions at a particular site, they ignore the information regarding the duration of the shaking. Strong-motion duration, however, has been shown to affect structural collapse fragility and, as a result, collapse risk (Raghunandan et al., 2015; Chandramohan et al., 2016b; Fairhurst et al., 2019). Recent studies by the authors on steel and reinforced concrete (RC) moment frames found that as ground motion duration increases, not only do structures tend to collapse at lower intensities but also at smaller deformations (Bhanu et al., 2021; Bhanu et al., 2023b). The deformations associated with collapse, termed as dynamic deformation capacity (DDC), were found to be around 25% lower under a long duration set as compared to a spectrally equivalent short duration set for both kinds of frames.

NZS 1170.5 aims to implicitly consider the higher damage potential from long duration shaking for short period structures (0.0 s - 0.5 s) by using a "magnitude-weighting" approach where earthquakes of magnitude (M_W) less than 7.5 are given a lower weighting. Tarbali and Bradley, 2016, however, demonstrated that this approach of implicit consideration of duration and cumulative effects via causal parameters such as magnitude is not reliable. There can be a few potential avenues to explicitly incorporate the effect of duration in structural design by adjusting the design parameters of strength, ductility or deformation limits. This study explores one of these avenues and employs the relationship between structural dynamic deformation capacity and the 5-75% significant duration, $Ds_{5\text{-}75}$ (Trifunac and Brady, 1975), to highlight recent proposals from Bhanu et al., 2023a for a simple method to account for the mean duration of ground motions anticipated at a site in NZS 1170.5. The benefits of the proposed method are expected to be uniform and acceptable levels of collapse risk for structures designed at sites experiencing different duration ground motions. This is verified by conducting hazard-consistent collapse risk assessment of a case-study steel moment frame designed for a site in Nelson and its collapse risk compared for structural designs with and without duration considerations.

2 PROPOSED METHOD TO INCORPORATE "DURATION" IN NZS 1170.5

NZS 1170.5 aims to satisfy internationally acceptable levels of collapse and fatality risks by achieving an adequately low level of collapse risk at the ultimate limit state (ULS), which is verified for earthquake motions with a return period of 500-years (typically) or more (for regions of low seismicity) (Standards New Zealand, 2004). This comes through "a high degree of reliability of achieving the strength and ductility values that are assumed" and are expected to be maintained at sufficient levels at higher intensities. Therefore, as explicit collapse design criteria do not exist in NZS 1170.5, the modifications to control the collapse risk in the existing guidelines are introduced at the ULS level in this study. Past studies have shown that even though structural systems are not observed to be affected by duration at the design level, in terms of peak deformation and force demands, their reduced ultimate deformation capacity or ductility under long duration records creates a lower margin of safety against collapse (Barbosa et al., 2017; Bhanu et al., 2023b). The aim of this study is to propose a method to also bring that safety margin to code-intended levels for sites expecting long duration motions.

In Bhanu et al., 2021 and Bhanu et al., 2023b, the authors evaluated the dynamic deformation capacity (DDC) of 10 RC and 9 steel moment frames respectively, under 88 ground motions of varying duration in the range 1 s < $Ds_{5\text{-}75}$ < 80 s. The DDC of a structure is the largest storey drift ratio demand that could be sustained without collapsing. The DDC of the moment frame buildings analysed in the two studies was found to reduce with increasing $Ds_{5\text{-}75}$ following a bilinear trend described by Equation 1.

$$ln\,DDC = \begin{cases} c_0 + \varepsilon & ; Ds_{5-75} \leq D_c \\ a(ln\,Ds_{5-75}) + c_1 + \varepsilon & ; Ds_{5-75} > D_c \end{cases} \tag{1}$$

where c_0, c_1, and a are regression coefficients, and ε is the residual error term. a represents the slope of the trend in DDC with $Ds_{5\text{-}75}$. D_c is the critical duration value below which duration is not expected to influence DDC.

A study on SDOF systems with Ibarra-Medina-Krawinkler (IMK) peak oriented and bilinear hysteretic models (Ibarra et al., 2005) also found DDC to follow a similar bilinear trend with $Ds_{5\text{-}75}$ (Bhanu 2022). Based on the regression analysis done on the DDC data for RC frames, steel frames and SDOF systems, D_c = 5 s was considered to be an appropriate choice.

Section 7.5 of NZS 1170.5 provides a design storey drift ratio limit at the ULS level, θ_{ULS}, of 2.5%. As discussed above, although code-based designs are, on average, not expected to exceed this limit under long duration records at intensities corresponding to the design response spectrum, they have a higher risk of collapse due to their lower apparent dynamic deformation capacity. The relationship presented in Equation 2 is proposed here to explicitly compensate for this effect of duration by adjusting θ_{ULS} based on the median $Ds_{5\text{-}75}$ of ground motions anticipated at the site. It should also be noted that based on the variability observed in the effect of duration on DDC and collapse risk over a range of frames in the previous studies, the proposed method is unlikely to achieve exactly the same level of collapse risk for designs corresponding to different duration targets. Nonetheless, the method is expected to reduce the variation in collapse risk amongst such designs and is a step in the right direction as shown later in this paper and in Bhanu et al., 2023a.

$$ln\ \theta_{ULS} = \begin{cases} \ln 2.5\% & ; Ds_{5-75} \leq 5\ s \\ a(ln\ Ds_{5-75}) + c_1 & ; Ds_{5-75} > 5\ s \end{cases} \qquad (2)$$

where coefficient a represents the slope of the relationship and c_1 is a function of a. $Ds_{5\text{-}75}$ here refers to the median $Ds_{5\text{-}75}$ of ground motions anticipated at the site, conditional on the 10% in 50-year exceedance probability of spectral acceleration at the fundamental period of vibration of the structure (T_1) and 5% damping, $S_a(T_1, 5\%)$.

Based on the results of previous studies (Bhanu et al., 2021; Bhanu et al., 2023b) and application of the proposed method to case-study buildings (Bhanu et al., 2023a), the value of a is proposed to be -0.15. Hence, the relationship proposed to adjust θ_{ULS} is presented in Equation 3 and Figure 1.

$$\theta_{ULS} = \begin{cases} 2.5\% & ; Ds_{5-75} \leq 5\ s \\ e^{-0.15(lnDs_{5-75})-3.448} & ; Ds_{5-75} > 5\ s \end{cases} \qquad (3)$$

where e is the natural exponent and approximately equals to 2.72.

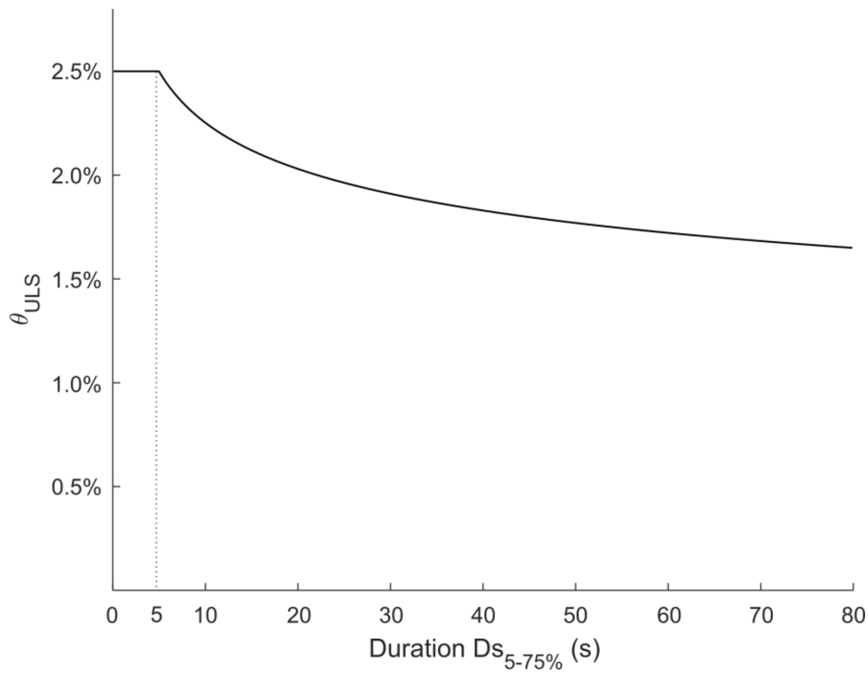

Figure 1: Proposed relationship to modify θ_{ULS} based on the median $Ds_{5\text{-}75}$ target for the site, as per Equation 3.

For the application of Equation 3 in practice, the median $Ds_{5\text{-}75}$ target for the site should be readily available to designers. Chandramohan et al. (2016a) described a procedure based on the generalised conditional intensity measure framework (Bradley 2010) to compute probability distributions of the durations of ground motions anticipated at a site, conditional on the exceedance levels of $S_a(T_1, 5\%)$. These values for the main population centres in New Zealand were presented in Chandramohan et al. (2018), although they are expected to be updated with the recent revision of the New Zealand National Seismic Hazard Model. Similar to the way the hazard factor, Z, is currently provided in NZS 1170.5 to account for ground motion intensity, introduction of $Ds_{5\text{-}75}$ hazard maps and tables for sites in NZ through a future amendment of NZS 1170.5 could be an easy way to provide the required $Ds_{5\text{-}75}$ targets to practitioners.

3 APPLICATION OF THE PROPOSED METHOD TO CASE-STUDY STEEL FRAME

A ductile 4-storey steel moment frame building (NEL04) designed for a site in Nelson (NZ) is used to demonstrate the application and benefits of the proposed method to explicitly account for the $Ds_{5\text{-}75}$ hazard. Nelson's seismic hazard has a significant (up to 60%) contribution from large-magnitude interface events leading to a noticeably higher median $Ds_{5\text{-}75}$ value as compared to many other major population centres in New Zealand and therefore, is an appropriate example of location where the effect of duration can significantly affect structural performance (Chandramohan et al., 2018). The considered frame was originally designed by Yeow et al., 2018 as per the NZS 1170 series (Standards New Zealand, 2004) and NZS 3404 (Standards New Zealand, 1997) for a site belonging to site class D in Christchurch, NZ. Given the similar 'Z' factors for Christchurch (0.30) and Nelson (0.27), the design was slightly modified to be used as a case-study building for site class D in Nelson. The frame has storey height of 4.5 m at the ground floor and 3.6 m on all other floors; it has three bays of 8.0 m width. The frame was designed with a ductility factor of 3.0, provided with reduced beam sections and expected to meet modern capacity design requirements. The fundamental period of vibration, T_1, for NEL04 was computed to be 1.2 s from eigenvalue analysis.

Three versions are carried out for the design of the frame: (i) original design (OD), (ii) target design (TD) and (iii) long duration design (LD). OD is the baseline design for a site in Nelson as per the current NZS 1170.5 guidelines, with θ_{ULS} of 2.5%. TD is the design achieved by modifying OD to satisfy the adjusted θ_{ULS} as per Equation 3 for the median $Ds_{5\text{-}75}$ target of Nelson. The $Ds_{5\text{-}75}$ targets for Nelson conditional on the 10% in 50 year exceedance probability of $S_a(1.2s, 5\%)$ for NEL04 is computed to be 14 s from the results of Chandramohan et al., 2018. Furthermore, to test the validity of the proposed method for a site expecting ground motions of median $Ds_{5\text{-}75}$ longer than that of Nelson, LD is the design carried out for a target $Ds_{5\text{-}75}$ of 44 s. The θ_{ULS} values for TD and LD are computed to be 2.14% and 1.80%, respectively, using Equation 3. The original design (OD) is modified for TD and LD by providing stronger beam and column sections to satisfy their respective design drift limits. The storey drifts of the frame at the ULS level were computed by conducting modal response spectrum analysis in line with the requirements of NZS 1170.5. Figure 2 shows the storey drift profiles of the frame at ULS for the three cases.

To verify the hypothesised benefits of the proposed method, hazard-consistent collapse risk assessments of the three designs of NEL04 were conducted. The structural reliability framework developed by Chandramohan et al., 2016a to obtain hazard-consistent collapse risk estimates by conducting incremental dynamic analysis (IDA) (Vamvatsikos and Cornell, 2002) using generic ground motion sets is employed here. Two-dimensional nonlinear structural models of the case-study buildings were developed in OpenSees (McKenna et al., 2006) to conduct IDA. The models consist of zero-length rotational plastic springs placed at the ends of a linear elastic element to simulate the non-linear response of the beams and columns. The modified Ibarra-Medina-Krawinkler (IMK) bilinear material model (Ibarra et al., 2005) was employed to define the hysteretic behaviour of the plastic hinges. The capacity of this hysteretic model to incorporate the in-cycle and cyclic deterioration of component strength and stiffness enables it to effectively capture the effect of duration (Chandramohan et al., 2016b). The parameters for the hysteretic model were characterised using the empirical equations provided by Lignos and Krawinkler, 2011. P-Δ effects on the frame were captured in the model through a pin-connected leaning column with the gravity load of the adjacent gravity frames acting on it and conducting large-displacement analysis. Rayleigh damping used with 2% critical damping was assigned to the periods corresponding to the first and third modes of the structures and to the linear elastic elements only (Charney, 2008). The fundamental modal periods of the three designs were found to be: (i) OD – 1.17 s, (ii) TD – 1.04 s and (iii) LD – 0.94 s. The nonlinear analyses were conducted using the central difference time integration scheme.

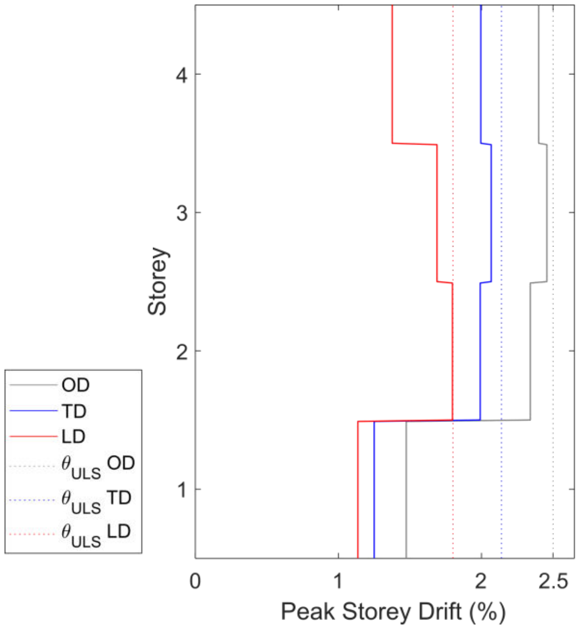

Figure 2: Comparison of storey drift demand profiles for NEL04 at the ULS level for the three designs: OD, TD and LD.

The ground motion set used in this study comprised of 88 generic records belonging to a wide range of duration, 1 s < $Ds_{5\text{-}75}$ < 80 s, that encompasses the median $Ds_{5\text{-}75}$ target values of interest. This ground motion set was originally assembled in Chandramohan et al., 2016b and more information regarding the set can be found there. IDA was conducted using the ground motion set to analyse the collapse performance of the frames in terms of collapse intensities. Equation 4 was then fit to the recorded collapse intensities, using the least squares method, to compute the median collapse intensities at the required $Ds_{5\text{-}75}$ targets.

$$\ln S_a(T_1, 5\%) \; at \; collapse = \; b_0 + \; b_{dur} ln Ds_{5-75} + \; \varepsilon \tag{4}$$

where b_0 and b_{dur} are regression coefficients and ε represents the error term.

Table 1 presents the median collapse intensities computed using the least-squares fitted Equation 4 for the different designs at the $Ds_{5\text{-}75}$ targets of interest. Observing the median collapse intensities for NEL04-OD, it can be seen that they reduce by 18% and 33% for $Ds_{5\text{-}75}$ targets of 14 s and 44 s, respectively, as compared to $Ds_{5\text{-}75}$ = 5 s. Similarly for TD and LD, it can be observed that the collapse intensities at longer duration targets are lower. These results again emphasise the motivation behind the proposed method in this study, as the collapse risk for the designs based on current NZS 1170.5 guidelines is expected to be noticeably higher at longer duration targets.

The primary purpose of the proposed modification in the design process, through Equation 3, has been to design structures with a uniform level of hazard-consistent collapse risk. Therefore, in order to compare the hazard-consistent collapse risk of the frames, their mean annual frequencies of collapse, $\lambda_{collapse}$, are estimated. At first, the collapse fragilities of the frames are computed as lognormal cumulative probability distribution functions with the median taken as their estimated median collapse intensity. Secondly, the seismic hazard curves for the site, Nelson, are obtained in terms of spectral intensity, $S_a(T_1, 5\%)$, as described in Chandramohan et al. (2018). Finally, $\lambda_{collapse}$ is evaluated by integrating the product of the collapse fragility curve and the derivative of the seismic hazard curve.

Table 1: Median collapse intensity, $S_a(T_1, 5\%)$, for the three designs of NEL04 computed at different $Ds_{5\text{-}75}$ targets. The highlighted values in red are for the targets corresponding to each design.

Design	$Ds_{5\text{-}75}$ = 5 s	$Ds_{5\text{-}75}$ = 14 s	$Ds_{5\text{-}75}$ = 44 s
OD	1.09 g	0.89 g	0.72 g
TD	1.85 g	1.55 g	1.28 g
LD	2.43 g	2.14 g	1.86 g

The estimated $\lambda_{collapse}$ of the frames at different targets are presented in Figure 3 and Table 2. In Figure 3, $\lambda_{collapse}$ for the designs based on current NZS 1170.5 guidelines, OD designs, indicate that the annual risk of collapse for the frame is around twice and three times at $Ds_{5\text{-}75}$ targets of Nelson and 44 s, respectively, as compared to a target of 5 s. Once again, these results show that if the same design is used for sites corresponding to different $Ds_{5\text{-}75}$ targets, the collapse risk of those similar frames at different sites will vary and be higher where longer duration ground motions are experienced.

Table 2: Mean annual frequency of collapse, $\lambda_{collapse}$, for the three designs of NEL04 computed at different $Ds_{5\text{-}75}$ targets. The highlighted values in red are for the targets corresponding to each design.

Design	$Ds_{5\text{-}75}$ = 5 s	$Ds_{5\text{-}75}$ = 14 s	$Ds_{5\text{-}75}$ = 44 s
OD	4.11×10^{-5}	7.16×10^{-5}	1.23×10^{-4}
TD	1.30×10^{-5}	2.25×10^{-5}	3.92×10^{-5}
LD	9.13×10^{-6}	1.36×10^{-5}	2.08×10^{-5}

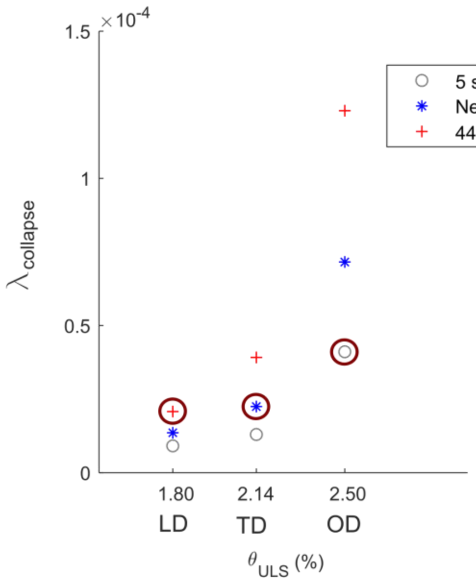

Figure 3: Mean annual frequency of collapse, $\lambda_{collapse}$, for the three designs of NEL04 computed at different $Ds_{5\text{-}75}$ targets, plotted against θ_{ULS}. $\lambda_{collapse}$ corresponding to the target $Ds_{5\text{-}75}$ value on which the design is based are highlighted using dark red circles.

The $\lambda_{collapse}$ for the frames, at a particular $Ds_{5\text{-}75}$ target, varies with the design drift limit, θ_{ULS}, in the order: LD < TD < OD, indicating that reducing θ_{ULS} at the design stage helps in reducing the collapse risk of the building. The hazard consistent (HC) $\lambda_{collapse}$ of the designs are $\lambda_{collapse}$ at $Ds_{5\text{-}75}$ target corresponding to the design and are highlighted using red circles in Figure 3. Comparing the HC-$\lambda_{collapse}$ for NEL04, it can be observed that the risk of collapse of the TD and LD designs is around 50% lower than OD.

The results presented here demonstrate that the collapse risk of structures at sites experiencing long duration records can be reduced by designing them for lower drift limits. For example, the HC-$\lambda_{collapse}$ values of the LD frame (θ_{ULS} = 1.8%) is 6 times smaller as compared to $\lambda_{collapse}$ of the OD frame (θ_{ULS} = 2.5%) at $Ds_{5\text{-}75}$ target of 44 s. The results further demonstrate that the variation in the annual risk of structural collapse at sites corresponding to different duration targets can also be reduced by modifying the design drift limits accordingly, as attempted through Equation 3 in this study. Although the estimated HC-$\lambda_{collapse}$ of the modified frame designs are not exactly the same as the original design, the variation in its level is noticeably reduced.

4 SUMMARY

This paper highlights a simple method, originally developed in Bhanu et al., 2023a, to explicitly incorporate the effect of duration in the seismic design process. A previously established relationships between the dynamic deformation capacity (DDC) of steel and RC frames and ground motion duration, $Ds_{5\text{-}75}$, is employed to adjust the design drift limit, taken here as the 2.5% storey drift limit at the ULS level, θ_{ULS}, given in the New Zealand standard NZS 1170.5. The proposed reduction in design drift limit, presented as Equation 3 and Figure 1, is expected to compensate for the reduced DDC and increased likelihood of collapse observed for structures at sites anticipating long duration shaking, thereby resulting in more uniform seismic risk for structures located at different sites.

To validate its hypothesised benefits, the proposed method was applied to a case-study steel frame building: a 4-storey building at a site in Nelson, New Zealand. Three versions of the design of the building were performed: (i) the original design based on the current θ_{ULS} of 2.5%, (ii) the target design based on θ_{ULS} adjusted as per Equation 3 for the median $Ds_{5\text{-}75}$ target of Nelson and (iii) the long duration design based on θ_{ULS} adjusted for a median $Ds_{5\text{-}75}$ target of 44 s. Hazard consistent collapse assessment of the three design versions of the building were performed by conducting IDA using 88 ground motions belonging to a wide range of duration, 1 s < $Ds_{5\text{-}75}$ < 80 s . The frame designs with lower θ_{ULS} were observed to have a lower $\lambda_{collapse}$. The variations in the HC-$\lambda_{collapse}$ values of the different design versions of each frame were observed to be less as compared to the $\lambda_{collapse}$ of the original designs at the considered $Ds_{5\text{-}75}$ targets. These findings suggest that modifying θ_{ULS} according to Equation 3 can effectively incorporate the effect of duration on structural collapse risk. More detailed information on the development and validation procedure of the proposed method can be found in Bhanu et al., 2023a.

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