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Drift-based risk-oriented method for the explicit consideration of ground motion duration in seismic design

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Abstract

This study proposes a method to explicitly account for ground motion duration in seismic design by modifying structural deformation capacity. An equation is presented to adjust the design drift limit prescribed in the New Zealand standard NZS 1170.5, based on the anticipated reduction in structural deformation capacity for ground motion durations longer than a critical value. The proposed relationship is used to derive designs corresponding to three duration targets each for two case-study steel moment frame buildings - a 4-storey and a 12-storey, located on a site in Nelson, New Zealand. Hazard-consistent collapse risk assessment of the design versions is conducted using a structural reliability framework employing incremental dynamic analysis. 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 steel frame buildings designed for different duration targets, compared to the existing approach. The method proposed in this study is simple and easy for practical applications and can be modified for other design codes and a range of structural typologies.

KEYWORDS

collapse risk, deformation capacity, ground motion duration, moment-frame buildings, seismic design

1 | INTRODUCTION

Current seismic codes around the world employ a uniform-risk spectrum[1] or uniform-hazard spectrum[2] based approach for structural design. Although the modern code-specified response spectra, generally derived through probabilistic seismic hazard analysis (PSHA), are used to estimate the mean intensity and frequency content of the anticipated ground motions for a given return period at a particular site, they ignore the information regarding the duration of the shaking. The value of strong-motion duration or significant duration, also derived through PSHA, has been shown to affect structural collapse fragility and, as a result, collapse risk[3–6]; this effect, however, is currently not explicitly accounted for in the design process. In the last decade while the number of studies demonstrating the relevance of incorporating duration in structural design has been large, studies proposing adequate methods to do so have been limited. Initial attempts explored the concept of using the distribution of secondary intensity measures, such as duration, conditional on commonly used primary intensity measures of spectral response to complement hazard curves.[7] More recently, Chandramohan et al.[4] and Liel et al.[8] derived methods to account for the mean duration hazard at a site by proportionally adjusting the design intensity level. As these studies were focused on the design practice in the USA, their proposed approach is limited to similar design codes that are based around a collapse limit state and cannot necessarily be used for other codes, such as the New Zealand seismic design standard, NZS 1170.5.[2] Recent studies by the authors have made the case to potentially incorporate the "duration" factor in design by adjusting the dependable deformation capacities of structures as an alternative approach that can be more widely applied across different design codes.[9,10] This study makes use of the findings of these studies to derive such a method.

There is an abundance of evidence in the literature now to confirm that the collapse risk of modern code-based structures is affected by ground motion characteristics other than spectral acceleration at the fundamental period, namely, spectral shape and duration (e.g., [11–14]). Chandramohan[12] showed through the nonlinear analyses of 51 ductile reinforced concrete (RC) frames that the variation in spectral acceleration at collapse can be up to over 80% controlled by these two parameters. This phenomenon leads to a non-uniform risk amongst structures designed using the same code but for different sites and the risk is more likely to exceed the acceptable levels where ground motions of long durations and flat spectral shapes are experienced. For example, the mean annual frequency of collapse of a modern RC frame in Eugene and Seattle were found to be around 60% and 30% underestimated, respectively, when hazard-consistent(HC) ground motions are not employed in collapse risk assessment; the two sites have significant hazard contribution from large magnitude interface earthquakes.[11] The higher collapse risk from such events also leads to significant increases in the predicted economic seismic losses.[15] This study focusses on the inclusion of only the "duration" characteristic in seismic design and spectral shape is considered out of scope. Recent studies by the authors on steel and 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.[9,10] 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 (0.0–0.5 s) only by using a "magnitude-weighting" approach where earthquakes of magnitude M less than 7.5 are given a lower weighting.[2] Tarbali and Bradley,[16] however, demonstrated that this approach of implicit consideration of duration and cumulative effects via causal parameters such as magnitude is not reliable. Similar to the methods proposed by Chandramohan[12] and Liel et al.,[8] 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 DDC and the 5%–75% significant duration, Ds_{5-75},[17] to propose a simple method to account for the mean duration of ground motions anticipated at a site in NZS 1170.5 by modifying the deformation capacity of the structure. 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 HC collapse risk assessments of two case-study steel moment frames designed for a site in Nelson and their collapse risk compared for structural designs with and without duration considerations.

2 | PROPOSED METHOD TO INCORPORATE "DURATION" IN NZS 1170.5

Section 7.5 of NZS 1170.5, the New Zealand standard for earthquake actions in structural design, provides a design storey drift limit at the ultimate limit state (ULS) level, θ_{ULS}, of 2.5%. This study proposes to reduce θ_{ULS} for sites with median $Ds_{5-75} > 5s$ using Equation (1) and Figure 1 to explicitly compensate for the effect of duration.

$$\theta_{ULS} = \begin{cases} 2.5\%, & \text{if } Ds_{5-75} \leq 5s \\ e^{-0.15(\ln Ds_{5-75})-3.448}, & \text{if } Ds_{5-75} > 5s \end{cases} \tag{1}$$

where e is the natural exponent and approximately equals to 2.72. Ds_{5-75} here refers to the median Ds_{5-75} of ground motions anticipated at the site, conditional on the 10% in 50 year exceedance probability of spectral acceleration, $S_a(T_1, 5\%)$.

The key steps of the proposed method to explicitly account for ground motion duration in NZS 1170.5 are summarised in Figure 2. For the application of the method in practice, the median Ds_{5-75} target for the site should be readily available

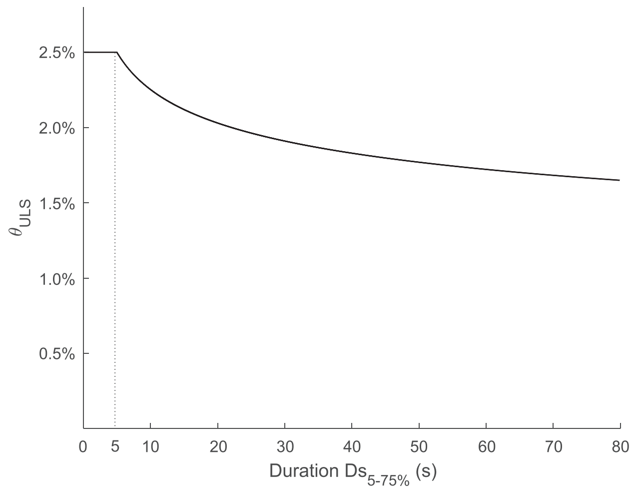

FIGURE 1 Proposed relationship to modify θ_{ULS} based on the median Ds_{5-75} target for site, as per Equation (1).

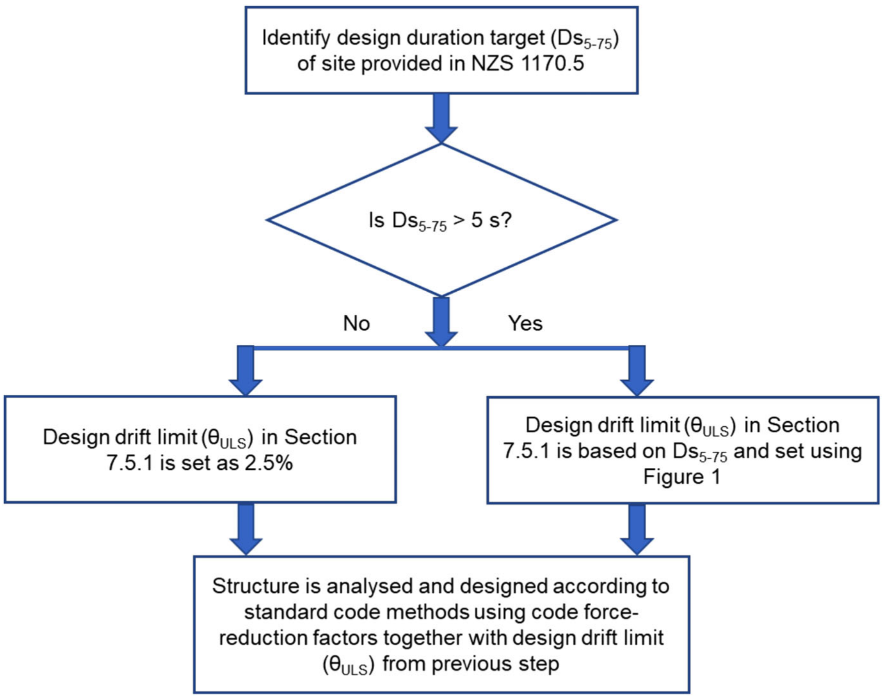

FIGURE 2 Summary of the proposed method to explicitly account for ground motion duration in NZS 1170.5.

to designers. Although duration targets are not currently provided in the design code, introduction of Ds_{5-75} 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_{5-75} targets to practitioners. Chandramohan et al.[11] described a procedure based on the generalised conditional intensity measure framework[18] to compute probability distributions of the durations of ground motions anticipated at a site, conditional on the exceedence levels of $S_a(T, 5\%)$. These values for the main population centres in New Zealand were presented in Chandramohan et al.,[19] although they are expected to be updated in the near future with the revision of the New Zealand National Seismic Hazard Model.

2.1 | Derivation of the proposed method

Unlike the US code - ASCE 7-16,[20] NZS 1170.5 is not based around a collapse limit state and rather on the ULS, which is verified for earthquake motions with a return period of 500-years (typically) or more (for regions of low seismicity).[2]

This, however, does not imply that internationally acceptable levels of collapse and fatality risks are not satisfied by NZS 1170.5 based designs. Instead, it expects to meet those levels by achieving a adequately low level of collapse risk at the ULS. 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.

The effect of duration on peak structural response metrics (for e.g., peak drift, peak acceleration, peak force/moment demands) is not evident at the ULS level intensities.[10,21,22] For example, only one out of nine steel frames analysed in Bhanu et al.,[10] was observed to have significantly larger drifts under long duration records as compared to short duration ones at ULS level. Duration does, however, affect cumulative demands such as hysteretic energy dissipated, cumulative inelastic strains, damage indices, and so forth. [21, 22]. The larger cumulative demands under long duration records lead to a higher strength and stiffness degradation, ultimately causing a full strength loss at relatively lower drifts and intensities.[9,10,22] Therefore, even though structural systems are not observed to be affected by duration at the design level, their reduced deformation capacity or ductility under long duration records creates a lower margin of safety against collapse. 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.[9] and Bhanu et al.,[10] the authors evaluated the DDC of 10 RC and 9 steel moment frames respectively, under 88 ground motions of varying duration in the range $1\,s < Ds_{5-75} < 80\,s$. The DDC of a structure is the largest storey drift demand that could be sustained without collapsing during incremental dynamic analysis.[23] The DDC of the moment frame buildings analysed in Bhanu et al.[9] and Bhanu et al.[10] was found to reduce with increasing Ds_{5-75} following a bilinear trend described by Equation (2).

$$\ln \mathrm{DDC} = \begin{cases} c_0 + \epsilon, & \text{if } Ds_{5-75} \leq D_c \\ a(\ln Ds_{5-75}) + c_1 + \epsilon, & \text{if } Ds_{5-75} > D_c \end{cases} \tag{2}$$

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_{5-75}. D_c is the critical duration value below which duration is not expected to influence DDC.

A study on SDOF systems with IMK peak-oriented and bilinear hysteretic models[24] also found DDC to follow a similar bilinear trend with Ds_{5-75}.[25] Based on the regression analysis done on the DDC data for RC frames, steel frames and SDOF systems in [9, 10, 25], $D_c = 5$ s is considered to be an appropriate choice.

Section 7.5 of NZS 1170.5 provides a uniform design storey drift 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 DDC. The relationship presented in Equation (3) is proposed in this study to explicitly compensate for this effect of duration by adjusting θ_{ULS} based on the median Ds_{5-75} of ground motions anticipated at the site. Ds_{5-75} is chosen to be the duration metric based on the suggestions of Chandramohan et al.[4], which found it to be well correlated to structural response. Equation (3) is based on the DDC versus Ds_{5-75} relationship observed in the previous studies by the authors.[9,10] The hypothesis that reducing the design storey drift limit will result in a reduction in the collapse risk is based on the findings of previous studies such as Gokkaya et al.[26] and Koopaee.[27] Further, it is assumed here that reducing θ_{ULS} by the same degree as DDC is found to reduce with Ds_{5-75} will result in uniform levels of collapse risk for the designs. In general, it is believed that reducing θ_{ULS} requires stronger and stiffer members in the design, which will have a higher hysteretic energy dissipation capacity and therefore reduced effects of cyclic degradation. Designs with lower θ_{ULS} are, as a result, expected to have higher DDC and reduced collapse risk. These assumption are supported by intuition and verified by the analyses conducted by the authors. 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 Ds_{5-75} 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.

$$\ln \theta_{ULS} = \begin{cases} \ln 2.5\%, & \text{if } Ds_{5-75} \leq 5s \\ a(\ln Ds_{5-75}) + c_1, & \text{if } Ds_{5-75} > 5s \end{cases} \tag{3}$$

where coefficient a represents the slope of the relationship and c_1 is a function of a as shown in Equation (4). Ds_{5-75} here refers to the median Ds_{5-75} of ground motions anticipated at the site, conditional on the 10% in 50 year exceedance probability of spectral acceleration, $S_a(T_1, 5\%)$.

TABLE 1 Source-specific conditional median target Ds_{5-75} values for Nelson and their corresponding percentage contributions to the total seismic hazard from each type of seismic source (indicated in parentheses), conditional on the 10% in 50-year exceedance probability of $S_a(T, 5\%)$ for a range of periods, T. The weighted averages are computed in logarithmic space.[19].

	Median target Ds_{5-75} for Nelson			
$T(s)$	Crustal earthquakes	Interface earthquakes	Intraslab earthquakes	Weighted average
0.5	6.3 s (44%)	22.2 s (25%)	7.4 s (31%)	9.1 s
1.0	8.1 s (46%)	23.6 s (47%)	8.9 s (7%)	13.5 s
2.0	10.2 s (29%)	23.9 s (60%)	10.2 s (11%)	17.0 s
3.0	12.3 s (41%)	24.3 s (56%)	10.9 s (3%)	17.9 s

$$c_1 = \ln 2.5\% - a \ln 5 \tag{4}$$

a for RC frames analysed in Bhanu et al.[9] was recorded in the range $-0.08 \leq a \leq -0.21$ with a mean of -0.15. The range of a recorded for steel frames was relatively wider, $-0.06 \leq a \leq -0.31$, with a mean of -0.19.[10] For SDOF systems a was recorded, on average, to be -0.13 for both bilinear and peak-oriented hysteresis over a wide range of periods T, $0.5s \leq T \leq 4s$.[25] Considering these results and the investigations conducted by the authors, a is initially proposed to be -0.15 for both steel and RC moment frame buildings; the validity of the value of a is tested in the next section of this paper when the proposed method is applied to case-study designs. Hence, the preliminary relationship proposed to adjust θ_{ULS} is presented in Equation (1) and Figure 1.

3 | APPLICATION OF THE PROPOSED METHOD TO CASE-STUDY STEEL FRAMES

3.1 | Case-study frames and designs

Two ductile steel moment resisting frame buildings, a 4-storey (NEL04) and a 12-storey (NEL12) building, designed for a site in Nelson (NZ), are used to demonstrate the application and benefits of the proposed method to explicitly account for the Ds_{5-75} hazard. Nelson's seismic hazard has a significant (up to 60%) contribution from large-magnitude interface events leading to a noticeably higher median Ds_{5-75} value as compared to many other major population centres in New Zealand and therefore, is considered to be an appropriate example of location where the effect of duration can significantly affect structural performance.[19] The source-specific conditional median target Ds_{5-75} values for Nelson and their corresponding percentage contributions to the total seismic hazard from each type of seismic source, conditional on the 10% in 50 year exceedance probability of $S_a(T, 5\%)$, were obtained from Chandramohan et al.[19] and are summarised in Table 1. The median Ds_{5-75} targets for Nelson, as observed from Table 1, imply that θ_{ULS} should be reduced at the design stage to compensate for the increased collapse risk from long duration records.

The considered frames were originally designed by Yeow et al.[28] as per the NZS 1170 series[2] and NZS 3404[29] 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 designs were slightly modified to be used as case-study buildings for site class D in Nelson. Both frames have storey heights of 4.5 m at the ground floor and 3.6 m on all other floors. The 4-storey and 12-storey frames have three and four bays, respectively, with bay widths of 8.0 m each. The frames were designed with a ductility factor, μ, of 3.0; they were provided with reduced beam sections (RBS) and expected to meet modern capacity design requirements. The fundamental period of vibration, T_1, for NEL04 and NEL12 are computed to be 1.2 and 2.2 s, respectively, from eigenvalue analysis.

Two versions are carried out for the design of each frame: (i) original design (OD) and (ii) target design (TD). 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, on the other hand, is the design achieved by modifying OD to satisfy the adjusted θ_{ULS} as per Equation (1) for the median Ds_{5-75} target of Nelson. The Ds_{5-75} targets for Nelson conditional on the 10% in 50 year exceedance probability of $S_a(1.2\,s, 5\%)$ and $S_a(2.2\,s, 5\%)$ for NEL04 and NEL12, respectively, are computed to be 14 and 17 s. These values are estimated by interpolating the data presented in Table 1 at the periods of interest. Furthermore, to test the validity of the proposed method for a site expecting ground motions of median Ds_{5-75} longer than that of Nelson, a third version of frame design, termed a long duration design (LD), is carried out for a target Ds_{5-75} of 44 s. 44 s is the geometric mean Ds_{5-75} of the long duration ground motion set employed in the previous studies by the authors, such as Bhanu et al.[9] and Bhanu et al.,[10] as well as in this

TABLE 2 θ_{ULS} considered for the three design versions computed using Equation (1). (Median Ds_{5-75} targets for the designs are indicated in parentheses).

	θ_{ULS}	
Design	**NEL04**	**NEL12**
OD	2.50% (5 s)	2.50% (5 s)
TD	2.14% (14 s)	2.08% (17 s)
LD	1.80% (44 s)	1.80% (44 s)

Abbreviations: LD, long duration design; OD, original design; TD, target design.

study. In other words, the LD design is for a hypothetical scenario where Nelson has a Ds_{5-75} target of 44 s. Therefore, the site for LD is considered to be similar to site class D in Nelson in all other aspects. Table 2 presents the Δ_{ULS} values considered for the three design versions, computed using Equation (1). Based on its θ_{ULS} of 2.5%, OD is assumed here to be designed for a target Ds_{5-75} of 5 s which approximately corresponds to an earthquake scenario of magnitude 6.5 at 30 km rupture distance.[30]

The original designs are modified for TD and LD by providing stronger beam and column sections to satisfy the design drift limits of Table 2. The storey drifts of the frames at the ULS level were computed by conducting modal response spectrum analysis and applying the recommended drift modification factors and P-Δ considerations as per section 7 of NZS 1170.5. The drifts obtained were further amplified by 10% to account for the reduction in stiffness from RBS cutouts[31] and an additional 16% as an estimate of any torsional effects. The final designs chosen were the ones that satisfied their corresponding θ_{ULS} criteria closely.

3.2 | Numerical modelling

Two-dimensional nonlinear structural models of the case-study buildings were developed in OpenSees[32] to conduct incremental dynamic analysis (IDA)[23] for collapse risk assessment. 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. Following the approach of a number of previous studies investigating the effect of duration on structural response (e.g., [3, 4, 22]), the modified Ibarra-Medina-Krawinkler (IMK) bilinear material model,[24] available as "IMKBilin" in OpenSees, 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.[4] The parameters for the hysteretic model were characterised using the empirical equations provided by Lignos and Krawinkler.[33] A limitation of the lumped plasticity model employed here is that it does not capture axial-flexural interaction effects. There is not much information available in the literature regarding the effects of such a modelling assumption on the observed influence of duration and it remains to be a topic for future investigation; it is assumed here that such a limitation will uniformly affect results recorded under different duration ground motions. The beam-column joint panel was modelled as a parallelogram shaped shear panel, "Joint 2D" in OpenSees, with beam and column elements connected to the midpoints of its sides. The possibility of the shear yielding of the joint panels was modelled using the shear distortion relationship developed by Krawinkler.[34] P-Δ effects on the frames 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. The seismic masses and gravity loads of the frames were uniformly applied to their beam-column joints. 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.[35] The nonlinear analyses were conducted using the central difference time integration scheme with time-steps small enough to satisfy the stability criteria. Although the use of this explicit scheme can be computationally intensive, it has been shown to be more robust against numerical non-convergence that can lead to premature simulation of collapse.[12,36]

3.3 | Implications of the proposed method on building design

Figure 3 shows the final storey drift profile of the frames at ULS for the three cases confirming that the designs achieve their target drift limits. Comparison of the sections used as beams and columns in the three designs is presented in Tables 3

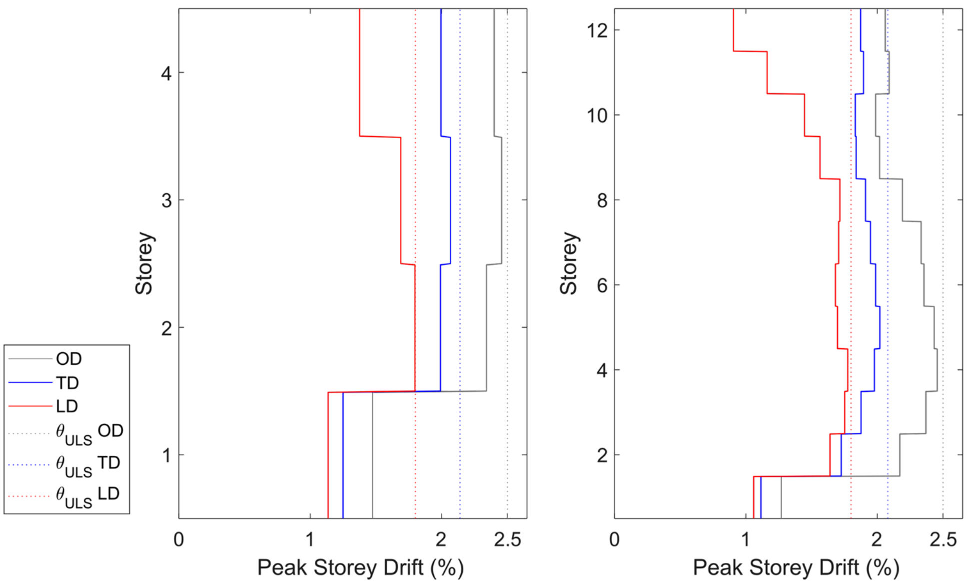

FIGURE 3 Comparison of storey drift demand profiles of NEL04 and NEL12 at the ULS level for the three designs: OD, TD and LD. LD, Long duration design; OD, original design; TD, target design.

TABLE 3 Sections used as beams and columns for the three design versions of NEL04.

Storey	Beams			Columns		
no.	OD	TD	LD	OD	TD	LD
4	410UB53.7	410UB53.7	530UB92.4	900WB175	900WB175	900WB218
3	530UB82.0	610UB101	610UB125	900WB175	900WB175	900WB218
2	610UB101	610UB113	610UB125	900WB175	900WB218	900WB257
1	610UB113	610UB125	610UB125	900WB175	900WB218	900WB257

Abbreviations: LD, Long duration design; OD, original design; TD, target design.

and 4 for the 4-storey and 12-storey frames, respectively. The fundamental modal periods of the three designs of each frame are indicated in Table 5 and expectedly, designs corresponding to longer duration targets have shorter periods. The fundamental period of both the frames reduced by approximately 10% and 25% to satisfy the drift limits for duration targets of 14-17 s and 44 s respectively, compared to a 2.5% drift limit. The weight of structural steel used in the modified designs also increased by approximately 10% and 30% for TD and LD respectively. Stiffer designs can lead to higher floor acceleration demands which should be accounted for by the designers for other aspects such as seismic restraints for non-structural elements, floor diaphragm reinforcement, and so forth. Therefore, it is expected that designs corresponding to long duration targets can have relatively higher costs for structural and non-structural systems. However, an increase in cost for these designs is expected to provide a level of seismic resilience similar to that intended by the code for designs at short duration target sites. Figure 4 compares the static pushover response of the different designs for the two frames. The peak base shear recorded for the LD frame, designed for an almost nine times higher Ds_{5-75} target, is 60% and 40% higher for the 4-storey and 12-storey frames respectively, as compared to the OD frame.

3.4 | Incremental dynamic analysis

In order to verify the hypothesised benefits of the proposed method, HC collapse risk assessments of the three designs of each case-study frame are conducted. Traditionally, such an assessment is performed through multiple stripe analysis (MSA)[37] which employs ground motion records matching the site-specific HC target distributions of the required characteristics, such as duration and spectral shape, at each intensity level. Availability and development of such site-specific ground motion sets, however, can be difficult and time-consuming, especially for sites with long duration targets. Chandramohan et al.[4] developed a structural reliability framework to obtain HC collapse risk estimates similar to MSA, but by

TABLE 4 Sections used as beams and columns for the three design versions of NEL12.

Storey no.	Beams OD	Beams TD	Beams LD	Columns OD	Columns TD	Columns LD
12	410UB53.7	410UB59.7	700WB150	1200WB249	1200WB249	1200WB278
11	530UB92.4	610UB101	700WB150	1200WB249	1200WB249	1200WB278
10	610UB113	610UB125	700WB150	1200WB249	1200WB249	1200WB278
9	700WB150	700WB173	700WB173	1200WB278	1200WB278	1200WB342
8	700WB150	700WB173	800WB168	1200WB278	1200WB278	1200WB342
7	700WB150	800WB168	800WB168	1200WB278	1200WB278	1200WB342
6	800WB146	800WB192	900WB218	1200WB342	1200WB342	1200WB392
5	800WB146	800WB192	900WB218	1200WB342	1200WB342	1200WB392
4	800WB168	800WB192	900WB257	1200WB342	1200WB392	1200WB423
3	800WB168	900WB218	900WB257	1200WB392	1200WB392	1200WB423
2	800WB168	900WB218	900WB257	1200WB392	1200WB392	1200WB423
1	800WB168	900WB218	900WB257	1200WB392	1200WB392	1200WB423

Abbreviations: LD, Long duration design; OD, original design; TD, target design.

TABLE 5 The fundamental period of vibration (T_1) of NEL04 and NEL12 for the three designs.

Design	T_1 NEL04	T_1 NEL12
OD	1.17 s	2.23 s
TD	1.04 s	1.97 s
LD	0.94 s	1.76 s

Abbreviations: LD, long duration design; OD, original design; TD, target design.

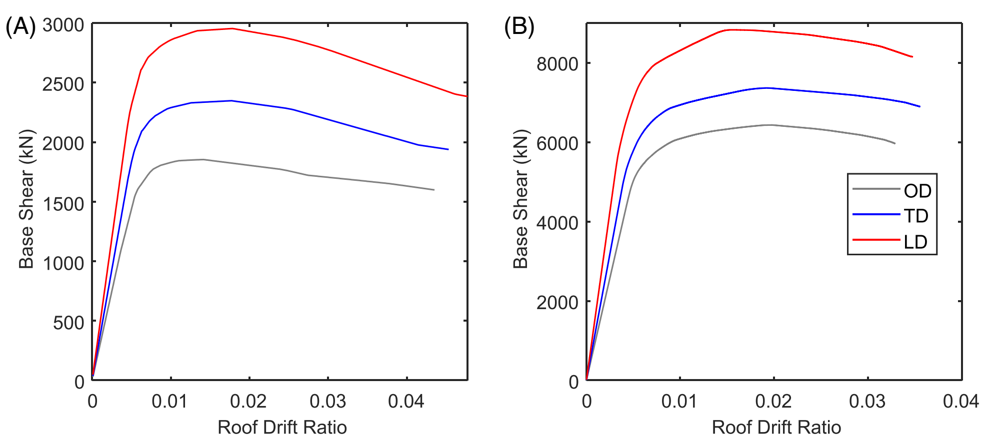

FIGURE 4 Static pushover response of the (A) NEL04 and (B) NEL12 frames for the three designs: OD, TD and LD. LD, Long duration design; OD, original design; TD, target design.

conducting IDA using generic ground motion sets. As this framework avoids the laborious process of MSA, it is employed in this study to conduct the required collapse assessment.

The ground motion set used in this study is comprised of 88 generic records belonging to a wide range of duration, $1\,s < Ds_{5-75} < 80\,s$, that encompasses the median Ds_{5-75} target values of interest. This ground motion set was originally assembled in Chandramohan[12] and contains 44 records of the FEMA P695[38] far-field set that are from shallow crustal events and have $Ds_{5-75} < 25\,s$. The other 44 other records are of relatively longer durations, from large magnitude subduction and crustal events such as 2011 Tohoku (Japan) and 2008 Wenchuan (China). The distribution of the Ds_{5-75} values and mean spectral response of the ground motion set are presented in Figure 5. Though the effect of spectral shape is not

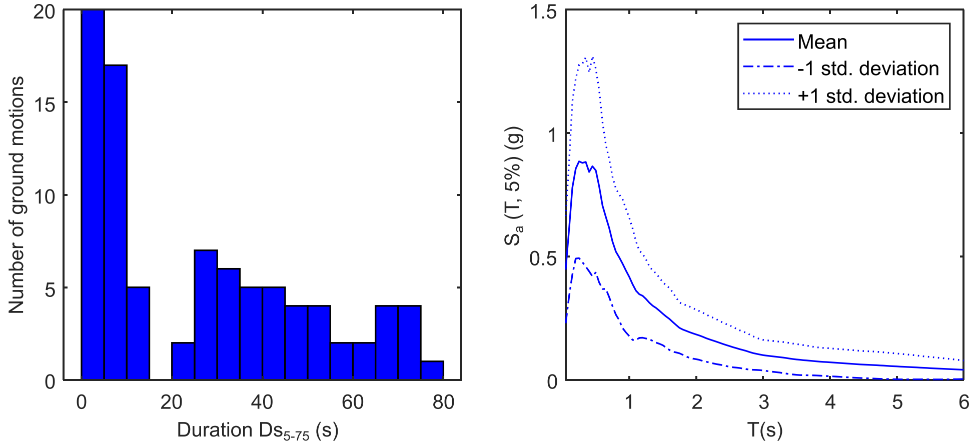

FIGURE 5 (A) Distribution of Ds_{5-75} of records in the ground motion set employed, and (B) their mean and 1 standard deviation spectral response.

considered in this study and S_aRatio[39] targets for the considered site are ignored in the collapse assessment, the ground motion set covers a wide range of S_aRatio values at the periods of interest.

IDA was conducted using the ground motion set to analyse the collapse performance of the frames in terms of collapse intensities and DDC. Since the accuracy of IDA results is inversely related to the intensity increments employed, fine $S_a(T_1, 5\%)$ increments of 0.02 were used and further reduced to 0.005 near the collapse intensity. Each IDA was conducted up to the point of collapse, which was identified as any storey drift in the frame reaching a threshold of 20%. The estimation of DDC can be affected by the deformation threshold chosen and this can disproportionately affect the results depending on the duration of the ground motion. Therefore, in spite of the fact that real life buildings will have lower deformation capacities, a high deformation threshold of 20% is used here to minimise any dependence of DDC on the value chosen.

3.5 | HC collapse risk assessment

The IDA results obtained are used to compute the collapse intensities and DDC of the frames as described previously in this paper and in Bhanu et al.[9] To compute the median collapse intensities at the required Ds_{5-75} targets, Equation (5) is fit to the recorded collapse intensities, using the least squares method. It is a modification of the original relationship proposed by Chandramohan[12] and estimates the variation in collapse intensity as a function of Ds_{5-75}. Previous studies by the authors have shown structural DDC to be unaffected by S_aRatio.[9,10,25] Furthermore, S_aRatio of the ground motion set employed was also confirmed to be uncorelated to its Ds_{5-75}. Due to the non-availability of real S_aRatio targets for Nelson and considering that the primary focus here is to investigate the effect of duration, S_aRatio is ignored in the collapse assessment process. Although the inclusion of real S_aRatio targets would provide a more accurate prediction of the median collapse intensity, the relative results, in terms of mean annual frequency of collapse associated with different Ds_{5-75} targets, are not expected to be much different without it. The approach followed here should instead give an estimate of the median collapse capacity for a S_aRatio target equal to the geometric mean S_aRatio of the ground motion set employed.

$$\ln S_a(T_1) \text{ at collapse} = b_0 + b_{dur} \ln Ds_{5-75} + \epsilon \tag{5}$$

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

Similarly, Equation (2) is fit to the recorded DDC of the frames for $D_c = 5$ s. The coefficients of the least-squares fit of Equations (5) and (2) for the frames are indicated in Table 6. Figures 6 and 7 plot the recorded collapse intensities and DDC, respectively, against Ds_{5-75} along with the fitted least-squares regression line for the three designs of NEL04 and NEL12. It can be observed from the figures that the DDC and collapse intensities follow a decreasing trend with increasing Ds_{5-75}. The only exception here is the NEL12-LD frame for which the correlation between collapse intensities and Ds_{5-75}

TABLE 6 Regression coefficients computed for the least-squares fit of Equation (2) and Equation (1) for the case-study frames. CI refers to collapse intensity.

	DDC vs. Ds_{5-75}, Equation (2)			CI vs. Ds_{5-75}, Equation (1)	
Frame	c_0	c_1	a	b_0	b_{dur}
NEL04-OD	−3.05	−2.74	−0.20	0.38	−0.19
NEL04-TD	−2.33	−1.93	−0.25	0.89	−0.17
NEL04-LD	−2.10	−1.70	−0.25	1.08	−0.12
NEL12-OD	−2.79	−2.16	−0.40	0.00	−0.17
NEL12-TD	−2.54	−1.70	−0.52	0.38	−0.21
NEL12-LD	−2.70	−2.47	−0.14	0.39	−0.05

Abbreviation: DDC, dynamic deformation capacity.

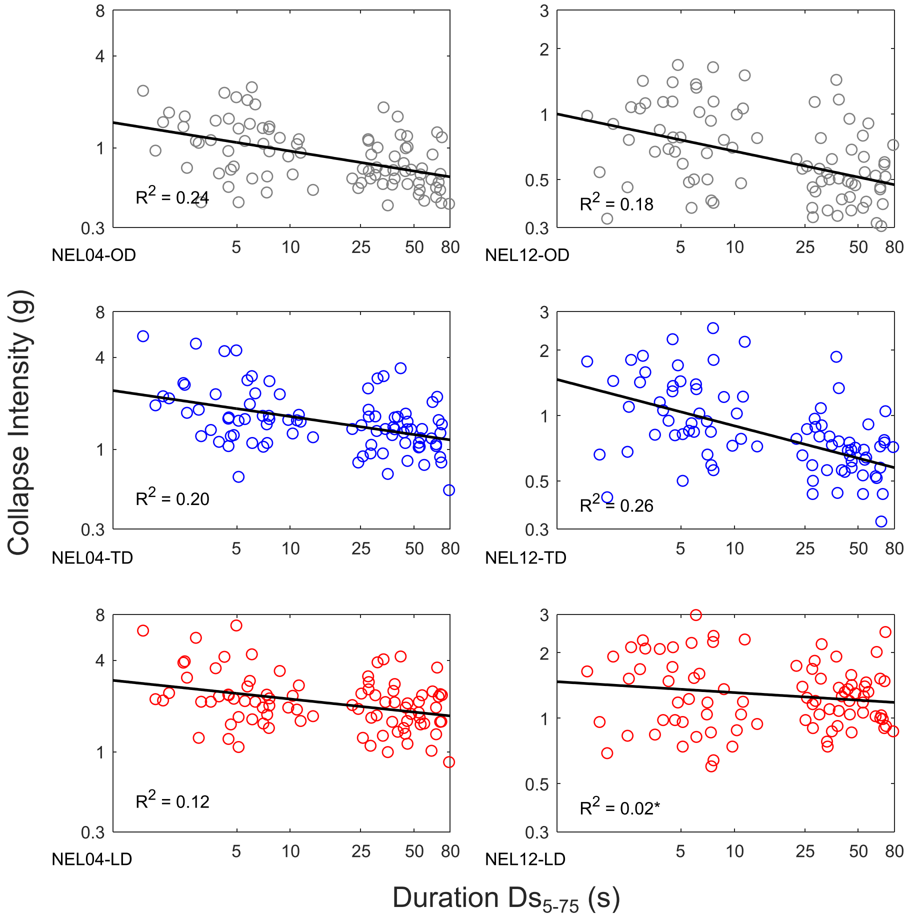

FIGURE 6 Log-log plot of collapse intensity ($S_a(T_1, 5\%)$ at collapse) versus Ds_{5-75} with the least-squares Equation (2) for the three designs of the NEL04 and NEL12 frames. **p*-value of the slope of the least-squares regression line is greater than 0.05.

was not found to be statistically significant indicating that the collapse intensities for this frame are not observed to vary with duration.

Table 7 presents the median collapse intensities computed using the least-squares fitted Equation (5) and using the coefficients from Table 6 for the different designs at the Ds_{5-75} targets of interest. The estimated median collapse intensity for the NEL12-LD frame at any Ds_{5-75} target is considered to be the geometric mean of all collapse intensities recorded from the ground motion set as the correlation with Ds_{5-75} was not found to be statistically significant. Observing the median collapse intensities for NEL04-OD, it can be seen that they reduce by 18% and 33% for Ds_{5-75} targets of 14 and 44 s, respectively, as compared to $Ds_{5-75} = 5$ s. Similarly, for NEL12-OD the median collapse intensities are 19% and 31% lower for Ds_{5-75} targets of 17 and 44 s, respectively. These results again emphasise the motivation behind the proposed method

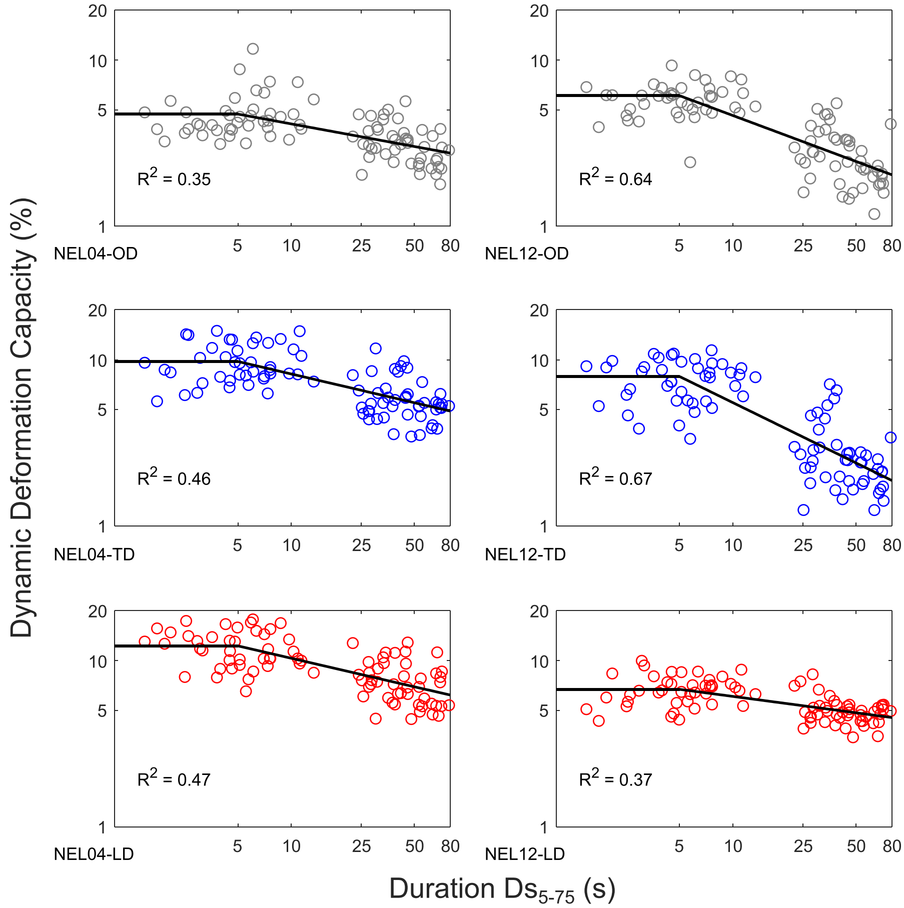

FIGURE 7 Log-log plot of DDC versus $Ds_{5-75}$with the least-squares Equation (2) for the three designs of the NEL04 and NEL12 frames. DDC, dynamic deformation capacity.

TABLE 7 Median collapse intensity, $S_a(T_1, 5\%)$, for the three designs of NEL04 and NEL12, computed at different Ds_{5-75} targets. The highlighted values in red are for the targets corresponding to each design.

	Median collapse intensity, $S_a(T_1,5\%)$					
	NEL04			NEL12		
Frame Design	Ds_{5-75} = 5 s	14 s	44 s	5 s	17 s	44 s
OD	1.09 g	0.89 g	0.72 g	0.76 g	0.62 g	0.52 g
TD	1.85 g	1.55 g	1.28 g	1.04 g	0.80 g	0.65 g
LD	2.43 g	2.14 g	1.86 g	1.29 g	1.29 g	1.29 g

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 HC collapse intensities for the designs, the collapse intensity corresponding to the target Ds_{5-75} on which the design is based, are highlighted in red in Table 7.

The primary purpose of the proposed modification in the design process, through Equation (1), has been to design structures with a uniform level of HC collapse risk. The median collapse intensities, presented in Table 7, for the three designs of each frame are in terms of spectral intensity at their respective fundamental periods of vibration, and therefore, cannot be compared directly. In order to compare the HC 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, μ, taken as their estimated median collapse intensity. The lognormal standard deviation, β, of the collapse fragility curves is assumed to be 0.6, based on the recommendations of FEMA P-695.[38] This value of β is expected to adequately account for the different modes of uncertainty contributing to the variability in collapse capacity, such as record-to-record, design and modelling related. Secondly, the seismic hazard curves for the site, Nelson, are obtained in terms of spectral intensity, $S_a(T_1)$. The data for these curves was obtained by performing PSHA

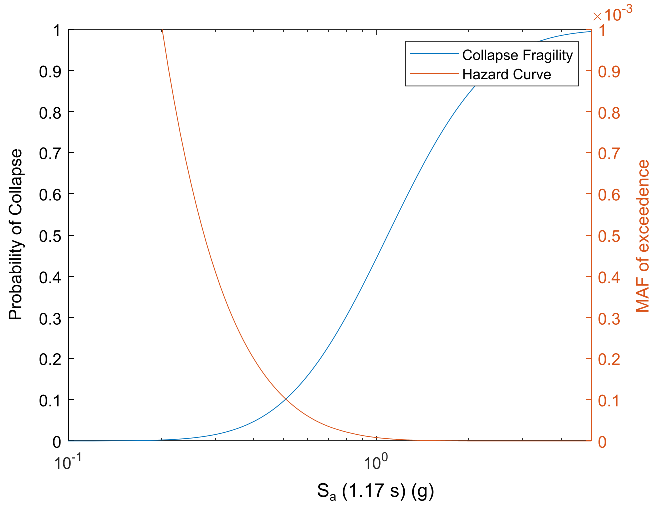

FIGURE 8 HC collapse fragility curve of the NEL04-OD frame along with its seismic hazard curve.HC, hazard consistent.

TABLE 8 Mean annual frequency of collapse, $\lambda_{collapse}$, of the three designs of the NEL04 and NEL12 frames at different Ds_{5-75} targets.

	$\lambda_{collapse}$ ($\times 10^{-5}$)					
	NEL04			NEL12		
Frame design	$Ds_{5-75} = 5$ s	14 s	44 s	5 s	17 s	44 s
OD	4.1	7.2	12.3	1.1	2.0	3.4
TD	1.3	2.3	3.9	0.6	1.5	2.8
LD	0.9	1.4	2.1	0.5	0.5	0.5

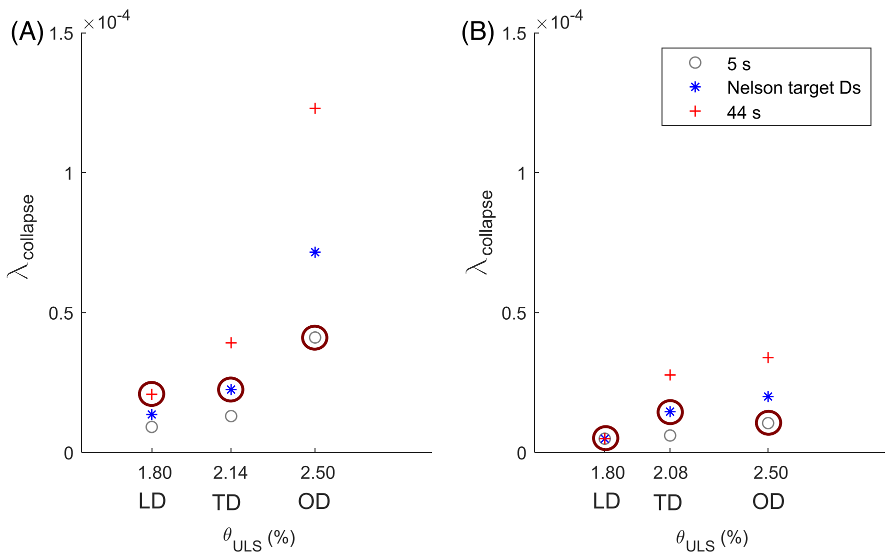

FIGURE 9 Mean annual frequency of collapse, $\lambda_{collapse}$, for the three designs of the (A) NEL04 and (B) NEL12 frames computed at different Ds_{5-75} targets, plotted against θ_{ULS}. The target Ds_{5-75} for Nelson is 14 and 17 s for the NEL04 and NEL12 frames, respectively. HC-$\lambda_{collapse}$, $\lambda_{collapse}$ corresponding to the target Ds_{5-75} value on which the design is based, are highlighted using dark red circles.

calculations using the OpenQuake Engine[40] and the implementation of the New Zealand national seismic hazard model by Horspool et al.,[41] 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. Figure 8 illustrates the HC collapse fragility curve along with the hazard curve for one of the analysed frames. The estimated $\lambda_{collapse}$ of the frames at different Ds_{5-75} targets are presented in Table 8 and Figure 9.

TABLE 9 Dynamic deformation capacity for the three designs of NEL04 and NEL12, computed at different Ds_{5-75} targets. The highlighted values in red are for the targets corresponding to each design.

	Dynamic deformation capacity					
	NEL04			NEL12		
Frame design	Ds_{5-75} = 5 s	14 s	44 s	5 s	17 s	44 s
OD	4.7%	3.9%	3.1%	6.1%	3.8%	2.6%
TD	9.8%	8.4%	5.9%	7.9%	5.0%	2.7%
LD	12.3%	10.6%	7.4%	6.7%	5.9%	5.0%

Abbreviations: LD, long duration design; OD, original design; TD, target design.

In Figure 9 and Table 8, $\lambda_{collapse}$ for the designs based on current NZS 1170.5 guidelines, OD designs, indicate that the annual risk of collapse for both frames is around twice and three times at Ds_{5-75} 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_{5-75} targets, the collapse risk of those similar frames at different sites will vary and be higher where longer duration ground motions are experienced. It can also be observed that $\lambda_{collapse}$ for the frames, at a particular Ds_{5-75} 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 $\lambda_{collapse}$ (HC-$\lambda_{collapse}$) of the designs are considered to be $\lambda_{collapse}$ at Ds_{5-75} target corresponding to the design and are highlighted in red in Table 8 and using red circles in Figure 9. Comparing the HC-$\lambda_{collapse}$ for the 4-storey frame, it can be observed that the risk of collapse of the TD and LD designs is around 50% lower than OD. Similarly for the 12-storey frame, HC-$\lambda_{collapse}$ for TD is 30% higher than OD but for the LD frame is almost half of 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 frames (θ_{ULS} = 1.8%) are six and seven times smaller as compared to the $\lambda_{collapse}$ values of the OD frames (θ_{ULS} = 2.5%) at Ds_{5-75} targets of 44 s for the 4-storey and 12-storey frames, respectively. 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 (1) 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 levels is observed to be noticeably reduced for the two frames. Precisely, while the $\lambda_{collapse}$ of the 4-storey OD varies up to 300% between Ds_{5-75} targets of 5–44 s, the HC-$\lambda_{collapse}$ of the two modified design versions is only 50% lower as compared to OD. For the 12-storey frame also, the $\lambda_{collapse}$ of OD varies up to 320% between Ds_{5-75} targets of 5–44 s; the HC-$\lambda_{collapse}$ of its modified design versions varies from −50% to 30% as compared to OD.

To verify if designs with a lower θ_{ULS} provide a higher DDC, the median DDC of the frames computed at different Ds_{5-75} targets of interest are also presented in Table 9 and Figure 10. For any particular design, the DDC can be observed to be lower at a longer duration target. For example, the NEL04-OD frame has 18% and 35% lower medain DDC at Ds_{5-75} targets of 14 and 44 s, respectively, as compared to Ds_{5-75} = 5 s. Similarly, the median DDC of the NEL12-OD frame are 38% and 58% lower at Ds_{5-75} targets of 17 and 44 s, respectively, as compared to DDC at a short duration target of 5 s. In general the DDC of the different designs are seen to be increasing in the order: LD > TD > OD, implying that the deformation capacity of frames designed using a lower θ_{ULS} is generally higher. The DDC of the OD frames, computed at Ds_{5-75} = 5 s, are compared to the DDC of the two modified designs, TD and LD, at their respective design Ds_{5-75} targets. These values are termed here as hazard consistent DDC, HC-DDC, and are highlighted using dark red circles in Figure 10. For the 4-storey frame, the TD and LD designs have around 80% and 60% higher HC-DDC as compared to the original design. For the 12-storey frame, the HC-DDC of the TD and LD versions are around 20% lower, respectively, as compared to that for OD. Although the modified designs did not achieve the same HC-DDC values as that of the original design, they are higher than the ones observed for the original design at their longer duration targets. A general indication of an increase in DDC with reducing θ_{ULS} is therefore observed here and is believed to be compensating for the increased likelihood of collapse of the frames at longer durations.

4 | DISCUSSION

The choice of −0.15 as the slope a in Equation (3) was based on the observation of the effect of duration on DDC in a couple of previous studies on steel and RC moment frames. Based on the results presented here, it can be said that this

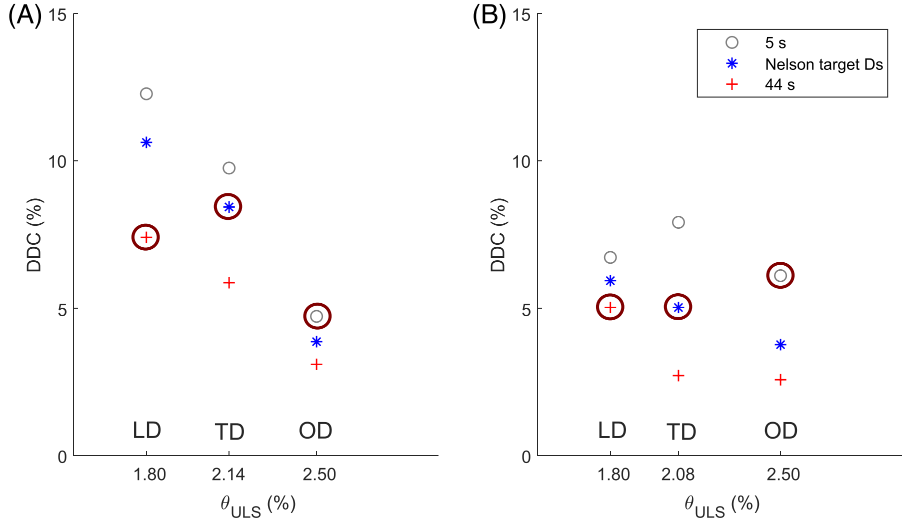

FIGURE 10 Median DDC of the three designs of the (A) NEL04 and (B) NEL12 frames computed at different Ds_{5-75} targets, plotted against θ_{ULS}. The target Ds_{5-75} for Nelson is 14 and 17 s for NEL04 and NEL12, respectively. HC-DDC, DDC corresponding to the target Ds_{5-75} value on which the design is based, are highlighted using dark red circles. DDC, dynamic deformation capacity; HC, Hazard consistent.

value is found to produce suitable reductions in collapse risk at different duration targets. The application of the proposed method resulted in a lower variability of collapse risk for designs corresponding to different duration targets. As observed in Bhanu et al.,[10] the effect of duration on steel frames can vary in a wide range for different frames and the extent of this effect can be hard to predict as no general trend was observed with structural properties. Therefore, proposing a value of a, in Equation (3), that leads to an exactly uniform level of collapse risk at different duration targets and also works for a wide range of structures is considered practically infeasible. Three out of four modified designs achieved a lower level of collapse risk as compared to the original design. While these results might suggest that the proposed slope is on a conservative side, the authors would justify its use considering that for the one other case the method was not found to be conservative enough. Nonetheless, the results in this study give an indication of the extent to which collapse risk can be controlled by using $a = -0.15$, which is observed to work well for the two steel frames analysed.

The application of the proposed method is shown for steel frames only in this study. However, based on a similar relationship observed between DDC and Ds_{5-75} in Bhanu et al.,[9] the results on RC frames are also expected to be similar. In another study, Koopaee[27] has demonstrated that reducing θ_{ULS} leads to a lower collapse probability for a range of New Zealand code-based RC frames, indicating that Equation (1) can also be used for such frames to compensate for the effect of duration. For other types of prevalent buildings, such as RC wall frames, braced steel frames and timber frames, although well defined relationships do not currently exist to estimate the effect of duration on DDC and collapse intensity, they have generally been observed to be affected by duration in a similar manner.[5,6,42,43] Extending the application of Equation (3) for these building typologies and identifying any other typologies less sensitive to duration could be a relevant topic for future work. Since the design of braced frame and wall frame buildings is usually not governed by drift limits, a strength based approach may be more appropriate for such structures. Future research can also investigate the comparative effectiveness of the different approaches to account for duration in design that is, drift and stiffness based against strength based. Similarly, though Equation (3) is proposed and validated here in accordance with the New Zealand NZS 1170.5 guidelines, a similar approach can be devised for other seismic design codes across the world that provide a design drift limit at a certain hazard level and are based on modern capacity design principles. For example, ASCE 7-16[20] prescribes a 2% drift limit at design level intensities, which can be modified following Equation (3) with an appropriate value of a to reduce the increased likelihood of collapse at sites with long duration targets. Another advantage of the proposed method is that its application is not limited to force-based design approaches and can also be extended to the displacement-based seismic design approach for structures.[44]

A limitation of this study is that the findings are applicable to the design of lateral load resisting systems of buildings only since the proposed method as well as its application are based on numerical analysis for such systems. The effect of gravity systems on the response of buildings to long duration motions has not been explored in the literature so far and therefore, the proposed method is not directly applicable to buildings whose collapse risk can be significantly affected by the components of such systems. Furthermore, the proposed method is based on the results of two-dimensional analysis on regular moment frame systems only. Therefore, the effect of bidirectional loading on 3-D systems as well as other realistic structural considerations such as stair systems, irregularities, and so forth. should be explored in a ground motion duration context by future studies to better incorporate these phenomena in design. It should also be noted that the collapse risk values presented in this study are by no means absolute, but rather indicative, and are only intended to indicate the impact of design process on the relative risk of the case-study buildings.

5 | CONCLUSIONS

This study developed a simple method to explicitly incorporate the effect of duration in the seismic design process. To do so, it employed previously established relationships between the DDC of steel and RC frames and ground motion duration, Ds_{5-75}, 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 (1) 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 the design of two case-study steel frame buildings: a 4-storey and a 12-storey building at a site in Nelson, New Zealand. Three versions of the design of each frame 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 (1) for the median Ds_{5-75} target of Nelson and (iii) the long duration design based on θ_{ULS} adjusted for a median Ds_{5-75} target of 44 s. HC collapse assessment of the three design versions of each frame were performed by conducting IDA using 88 ground motions belonging to a wide range of duration, $1\,s < Ds_{5-75} < 80\,s$. The frame designs with lower Δ_{ULS} were observed to have an increased DDC and a lower mean annual frequency of collapse, $\lambda_{collapse}$. 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_{5-75} targets. In general, the HC-$\lambda_{collapse}$ of the modified designs were found to be lower than that of the original designs for three out of four cases. These findings suggest that modifying θ_{ULS} according to Equation (1) can effectively incorporate the effect of duration on structural collapse risk.

Although derived in this study specifically for steel and RC moment frames and NZS 1170.5, the application of Equation (1) in its raw form, Equation (3), could be extended to other type of structures and design codes using a suitable value of a. Overall, the method proposed in this study is believed to be flexible, simple and easy for practical applications. A missing link, however, is the availability of median Ds_{5-75} targets for various sites. Such targets can easily be incorporated in future amendments of the design standards, in a way similar to the existing intensity based factors.

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DATA AVAILABILITY STATEMENT

The data that support the findings of this study are available from the corresponding author upon reasonable request.

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REFERENCES

1. Luco N, Ellingwood BR, Hamburger RO, Hooper JD, Kimball JK, Kircher CA. Risk-targeted versus current seismic design maps for the conterminous united states. In: *SEAOC 2007 Convention Proceedings*. Squaw Creek, California, 2007.
2. Standards New Zealand. *Nzs 1170.5 Structural Design Actions Part 5: Earthquake Actions-New Zealand*. Standards New Zealand; 2004.
3. Raghunandan M, Liel AB, Luco N. Collapse risk of buildings in the pacific northwest region due to subduction earthquakes. *Earthquake Spectra*. 2015;31(4):2087-2115. https://doi.org/10.1193/012114EQS011M

4. Chandramohan R, Baker JW, Deierlein GG. Quantifying the influence of ground motion duration on structural collapse capacity using spectrally equivalent records. *Earthquake Spectra*. 2016;32(2):927-950. https://doi.org/10.1193/122813eqs298mr2
5. Pan Y, Ventura CE, Liam Finn W. Effects of ground motion duration on the seismic performance and collapse rate of light-frame wood houses. *J Struct Eng*. 2018;144(8):04018112, 1-12. https://doi.org/10.1061/(ASCE)ST.1943-541X.0002104
6. Fairhurst M, Bebamzadeh A, Ventura CE. Effect of ground motion duration on reinforced concrete shear wall buildings. *Earthquake Spectra*. 2019;35(1):311-331. https://doi.org/10.1193/101117EQS201M
7. Iervolino I, Giorgio M, Galasso C, Manfredi G. Conditional hazard maps for secondary intensity measures. *Bull Seismol Soc Am*. 2010;100(6):3312-3319.
8. Liel AB, Luco N, Raghunandan M, Champion CP. Modifications to risk-targeted seismic design maps for subduction and near-fault hazards. In: *12th International Conference on Applications of Statistics and Probability in Civil Engineering*. Vancouver, Canada, 2015.
9. Bhanu V, Chandramohan R, Sullivan TJ. Influence of ground motion duration on the dynamic deformation capacity of reinforced concrete frame structures. *Earthquake Spectra*. 2021;37(4):2622-2637. https://doi.org/10.1177/87552930211033879
10. Bhanu V, Chandramohan R, Sullivan TJ. Influence of ground motion duration on the deformation demands and capacities of steel frame structures (in review). 2022.
11. Chandramohan R, Baker JW, Deierlein GG. Impact of hazard-consistent ground motion duration in structural collapse risk assessment. *Earthquake Eng Struct Dyn*. 2016;45(8):1357-1379. https://doi.org/10.1002/eqe.2711
12. Chandramohan R. *Duration of Earthquake Ground Motion: Influence on Structural Collapse Risk and Integration in Design and Assessment Practice*. Ph.D. thesis, Stanford University; 2016.
13. Marafi NA, Eberhard MO, Berman JW, Wirth EA, Frankel AD. Effects of deep basins on structural collapse during large subduction earthquakes. *Earthquake Spectra*. 2017;33(3):963-997. https://doi.org/10.1193/071916EQS114M
14. Chase RE, Liel AB, Luco N, Bullock Z. Hazard-consistent seismic losses and collapse capacities for light-frame wood buildings in california and cascadia. *Bull Earthquake Eng*. 2021;19(15):6615-6639. https://doi.org/10.1007/s10518-021-01258-y
15. Hwang S-H, Mangalathu S, Jeon J-S. Quantifying the effects of long-duration earthquake ground motions on the financial losses of steel moment resisting frame buildings of varying design risk category. *Earthquake Eng Struct Dyn*. 2021;50(5):1451-1468. https://doi.org/10.1002/eqe.3403
16. Tarbali K, Bradley BA. The effect of causal parameter bounds in psha-based ground motion selection. *Earthquake Eng Struct Dyn*. 2016;45(9):1515-1535. https://doi.org/10.1002/eqe.2721
17. Trifunac MD, Brady AG. A study on the duration of strong earthquake ground motion. *Bull Seismol Soc Am*. 1975;65(3):581-626.
18. Bradley BA. A generalized conditional intensity measure approach and holistic ground-motion selection. *Earthquake Eng Struct Dyn*. 2010;39(12):1321-1342. https://doi.org/10.1002/eqe.995
19. Chandramohan R, Horspool N, Bradley B. Duration of earthquake ground motion anticipated at sites in New Zealand. In: *New Zealand Society of Earthquake Engineering Annual Conference 2018*; 2018; Auckland, NZ.
20. ASCE. Minimum design loads for buildings and other structures. ASCE 7-16. American Society of Civil Engineers; 2016. https://ascelibrary.org/doi/abs/10.1061/9780784412916
21. Raghunandan M, Liel AB. Effect of ground motion duration on earthquake-induced structural collapse. *Struct Saf*. 2013;41:119-133. https://doi.org/10.1016/j.strusafe.2012.12.002
22. Barbosa AR, Ribeiro FL, Neves LA. Influence of earthquake ground-motion duration on damage estimation: application to steel moment resisting frames. *Earthquake Eng Struct Dyn*. 2017;46(1):27-49. https://doi.org/10.1002/eqe.2769
23. Vamvatsikos D, Cornell CA. Incremental dynamic analysis. *Earthquake Eng Struct Dyn*. 2002;31(3):491-514. https://doi.org/10.1002/eqe.141
24. Ibarra LF, Medina RA, Krawinkler H. Hysteretic models that incorporate strength and stiffness deterioration. *Earthquake Eng Struct Dyn*. 2005;34(12):1489-1511. https://doi.org/10.1002/eqe.495
25. Bhanu V. *Incorporating the Influence of Ground Motion Duration on Structural Deformation Capacity in Seismic Design*. PhD thesis. University of Canterbury, Christchurch, New Zealand; 2022.
26. Gokkaya BU, Baker JW, Deierlein GG. Quantifying the impacts of modeling uncertainties on the seismic drift demands and collapse risk of buildings with implications on seismic design checks. *Earthquake Eng Struct Dyn*. 2016;45(10):1661-1683. https://doi.org/10.1002/eqe.2740
27. Koopaee ME. *Seismic collapse probability prediction for loss optimisation seismic design (load)*. Ph.D. thesis. University of Canterbury; 2021.
28. Yeow T, Orumiyehei A, Sullivan T, MacRae G, Clifton G, Elwood K. Seismic performance of steel friction connections considering direct-repair costs. *Bull Earthquake Eng*. 2018;16(12):5963-5993. https://doi.org/10.1007/s10518-018-0421-x
29. Standards New Zealand. NZS 3404: 1997: Steel structures standard. Wellington, New Zealand, 1997.
30. Bommer JJ, Stafford PJ, Alarcón JE. Empirical equations for the prediction of the significant, bracketed, and uniform duration of earthquake ground motion. *Bull Seismol Soc Am*. 2009;99(6):3217-3233. https://doi.org/10.1785/0120080298
31. Cowie K. Design Example of Moment Resisting Seismic Frames with Reduced Beam Sections. Steel Construction of New Zealand; 2010.
32. McKenna F, Fenves G, Scott M. Opensees: Open system for earthquake engineering simulation. *Pacific Earthquake Engineering Research Center*. University of California; 2006. http://opensees.berkeley.edu
33. Lignos DG, Krawinkler H. Deterioration modeling of steel components in support of collapse prediction of steel moment frames under earthquake loading. *J Struct Eng*. 2011;137(11):1291-1302. https://doi.org/10.1061/(ASCE)ST.1943-541X.0000376
34. Krawinkler H. Shear in beam-column joints in seismic design of steel frames. *Eng J*. 1978;15(3):82-91.
35. Charney FA. Unintended consequences of modeling damping in structures. *J Struct Eng*. 2008;134(4):581-592. https://doi.org/10.1061/(ASCE)733-9445(2008)134:1(3)

36. Danielson KT, Akers SA, O'Daniel JL, Adley MD, Garner SB. Large-scale parallel computation methodologies for highly nonlinear concrete and soil applications. *J Comput Civil Eng*. 2008;22(2):140-146. https://doi.org/10.1061/(ASCE)0887-3801(2008)22:2(140)
37. Jalayer F. *Direct Probabilistic Seismic Analysis: Implementing Non-linear Dynamic Assessments*. Stanford University; 2003.
38. FEMA. Quantification of building seismic performance factors. US Department of Homeland Security, Federal Emergency Management Agency; 2009.
39. Eads L, Miranda ELignos, D,. Spectral shape metrics and structural collapse potential. *Earthquake Eng Struct Dyn*. 2016;45(10):1643-1659. https://doi.org/10.1002/eqe.2739
40. Pagani M, Monelli D, Weatherill G, et al. Openquake engine: an open hazard (and risk) software for the global earthquake model. *Seismol Res Lett*. 2014;85(3):692-702. https://doi.org/10.1785/0220130087
41. Horspool N, Abbott E, Canessa S, et al. Challenges and opportunities in developing a national seismic hazard and risk model with openquake for new zealand. *16th World Conference on Earthquake Engineering*. Santiago, Chile, 2017.
42. Hammad A, Moustafa MA. Modeling sensitivity analysis of special concentrically braced frames under short and long duration ground motions. *Soil Dyn Earthquake Eng*. 2020;128:105867. https://doi.org/10.1016/j.soildyn.2019.105867
43. Hammad A, Moustafa MA. Shake table tests of special concentric braced frames under short and long duration earthquakes. *Eng Struct*. 2019;200:109695. https://doi.org/10.1016/j.engstruct.2019.109695
44. Priestley MJN, Calvi MC, Kowalsky MJ. *Displacement-based Seismic Design of Structures*. IUSS Press; 2007.

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