Inter-storey Drift Limits for Buildings at Ultimate limit States

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ABSTRACT: Design engineers often use quasi-static procedures to analyse buildings and to determine the inter-storey drift demands rather than inelastic time history analysis procedures. For elastic procedures such as the modal response spectrum method, the NZS1170.5 loading standard suggests modification (or scaling) factors for simulating inter-storey drift in the post-elastic range corresponding to an ultimate limit state condition. In this study, the consistency between inter-storey drifts obtained from modal response analysis and inelastic time history analysis are verified, and suitable scale factors for design purposes are suggested.

Building models representing reinforced concrete and steel braced frames with various levels of ductility were analysed for a family of earthquake records matched to the design spectrum. The design of buildings was carried out to achieve a target drift ratio of 2.5% which is the maximum limit suggested in the standard. The inter-storey drift profiles for reinforced concrete buildings from inelastic time history analysis basically followed the inter-story drift profiles derived from the elastic method but exceeded them at upper storeys. In steel braced frames, the inelastic displacement was limited to the lower 1/3rd of building height only. Even though the inelastic methods resulted in higher inter-storey drifts compared to elastic methods, all values were within the prescribed code limit of 2.5%.

1 INTRODUCTION

Inter-story drift is the difference in lateral deflection between two adjacent stories of a building subjected to lateral loads. Elastic analysis or inelastic time history analysis (ITHA) methods can be used to determine inter-storey drifts. Elastic design procedures remain by far the most prevalent method of analysis with modal response analysis (MRA) being the norm for buildings over 4 stories or having periods greater than 1 second, with equivalent static analysis still being commonly used for low-rise buildings of regular plan and less than 4 stories. The current loading standard NZS1170.5 (Standards New Zealand 2004) requires that drifts obtained from elastic procedures be scaled to simulate the post-elastic deformations that can be achieved at ultimate limit state (ULS) conditions.

The aim of the current research was to confirm the published drift scaling factors, or to develop rational alternative scaling factors to justify compliance with the drift limits stated within the standard. It is also important that there be consistency between the ULS drifts calculated using inelastic time history analyses (considered to most accurately reflect the building behaviour) and those derived using elastic analysis techniques.

The study involved ten 2-D structural forms that were compliant with the inter-storey drift criteria stipulated in the loading standard and their respective material standards. The buildings were designed using MRA. The buildings were then analysed using ITHA techniques with selected earthquake records. Comparisons were made between the interstorey drifts calculated from each method, i.e. those from scaled elastic response methods (in all cases MRA was engaged for that phase) and those determined from ITHA to derive appropriate modifications to the current scale factors.

2 DEFLECTION RELATED SCALING FACTORS IN NZS 1170.5

Deflection and inter-storey drift values obtained from different structural analysis methods are bound to vary. For example, the equivalent static method over-predicts deflections and inter-storey drifts in the structure compared to the modal response spectrum method. To obtain comparative values, NZS 1170.5:2004 requires (in Clause 6.2.3) a deflection scale factor, K_d be used to reduce the values obtained by the equivalent static method. Similarly, a drift modification factor, k_{dm}, is included (in Clause 7.3.1.1) to enhance the inter-storey deflections that are usually under-predicted by elastic methods when compared to inelastic time history analytical methods. Note that a constant value of drift modification factor is applied at every storey level, and it depends only on the building height. The focus of the present study is to verify the current provisions related to drift modification factors in the loading standards with respect to different structural systems and for different seismicity regions.

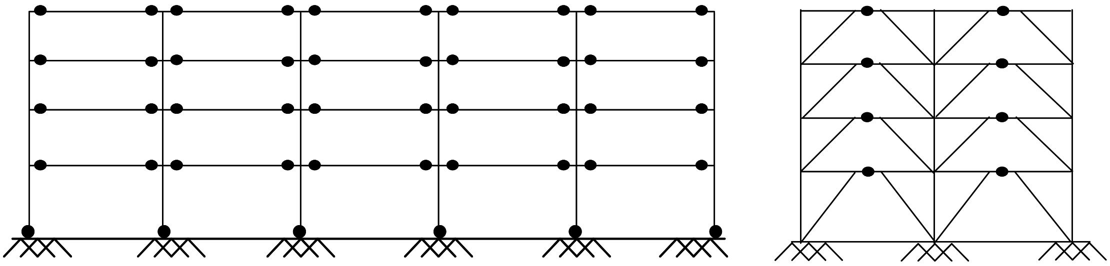
3 BUILDING DESIGNS

The study considered a suite of reinforced concrete (RC) and steel (ST) buildings representing two heights (3 and 10 occupied floors), two levels of seismicity (Wellington and Auckland), located in shallow and deep soil classes. All were designed using MRA. P-Delta effects were included using Method B in NZS 1170.5. Buildings with 3 occupied floors were considered to be representative of low-rise (~15 m) buildings and buildings with 10 occupied floors (>30 m) were considered to be representative of high-rise buildings, as listed in Table 1. Reinforced concrete buildings were configured with two seismic frames in the longitudinal direction and two shear walls in the transverse direction. In steel buildings, there were two seismic frames in one direction and two eccentrically braced frames (EBF) in the other direction. The buildings were designed to achieve a target inter-storey drift ratio of 2.5%, which is the maximum limit permitted by the loading standards. However, certain minimum requirements from materials standards with regard to the member sizes precluded the achievement of this limit and also controlled the strength and the ductility experienced. The influence of actual ductility experienced was not investigated within the scope of the present study. Complete details of building models can be obtained from elsewhere [Uma et al., 2009].

Table 1. Building details

Identification Name	Structural Material	No. of occupied floors	Structural Form	Location	Soil Class	Ductility		Period (T_1)
						ID	μ	
RC3WCD	RC	3	Frame	Wellington	C	D	6	1.1
RC3WCL	RC	3	Frame	Wellington	C	L	3	1.5
RC3ACL	RC	3	Frame	Auckland	C	L	3	2.15
RC10WCD	RC	10	Frame	Wellington	C	D	6	2.3
ST3WCD	ST	3	EBF	Wellington	C	D	5	0.7
ST3WCL	ST	3	EBF	Wellington	C	L	3	0.7
ST3WDD	ST	3	EBF	Wellington	D	D	5	0.7
ST3ACL	ST	3	EBF	Auckland	C	L	3	1.0
ST10WCD	ST	10	EBF	Wellington	C	D	5	1.75
ST10WCL	ST	10	EBF	Wellington	C	L	3	1.7
ST10WDD	ST	10	EBF	Wellington	D	D	5	1.6
ST10ACL	ST	10	EBF	Auckland	C	L	3	2.0

The scope of the work was limited to studying the responses of reinforced concrete frames and steel eccentrically braced frames in orthogonal directions of the buildings independently. This assumption is acceptable as per NZS 1170.5 for limited ductile (μ=3) and ductile (μ=6) buildings. Typical 2 dimensional non-linear models for the respective buildings are shown in Figure 1. The nonlinear dynamic simulations were performed using SAP2000. Cyclic inelastic deformations were modelled using nonlinear link elements. These elements were included in beams near the column faces for RC buildings and in the links of the frames in ST buildings. In RC buildings the column elements in the ground floor were modelled with fibre hinges to account for axial load-moment-interaction. P-Delta effects were considered in the analyses.



(a) Reinforced concrete frame (b) Eccentric braced frame

- Location where non-linearity is modelled

Figure 1: Typical 2-dimensional models for low-rise reinforced concrete frames and eccentrically braced steel frames.

For all buildings, inter-storey heights were assumed to be 4.5 m for the ground floor and 3.65 m for other floors. Reinforced concrete frames were designed with 5 bays each having a span of 7.5 m. EBF frames were modelled with two bays oft 8.5 m span.

4 MODELLING ASPECTS

The building models were provided with material properties based on expected strength, which is higher than dependable or specified strength. Nonlinear elements were included to reflect inelastic deformations. The expected strength of concrete was assumed to be 50% higher than the specified strength. The concrete stiffness used for design was increased by 30% to get the expected stiffness of the concrete. Reinforcing steel expected yield strengths were taken as being 15% higher than specified strengths as per standard practice. For steel buildings, the expected yield strength was taken as 1.15 times the specified strength. Strain-hardening effects were taken into account by the hysteretic models chosen. However, SAP2000 hysteretic models have a fixed-valued built-in stiffness degradation parameter. For concrete buildings, the member stiffness properties were modified to account for cracking. The effective stiffnesses for beams, internal columns and external columns were considered to be 33%, 50%, and 40% of respective gross sections. The above mentioned factors were in line with values used previously [Shelton, 2004].

The nonlinearity within the elements was modelled using nonlinear link (NL Link) elements to represent plastic hinges. An ‘NL Link’ element is a zero length element used to connect two coincident joints (Ref. SAP2000). They were included at the column faces at each end of all beams. Note that the NL link elements are not capable of modelling axial-moment (P-M) interaction, and so, the columns in frames where such interaction was significant were modelled with ‘fibre hinges’, which are recognised as an advanced feature of structural analysis. Therefore, the plastic hinges in columns at the base levels of reinforced concrete frames were modelled as fibre hinges. The length of plastic hinge zones was assumed to be 67% of the member depth. In eccentrically-braced frames, the column, brace and beam members were modelled using frame elements. The shear links were modelled as short beam members with normal flexural properties, but with shear properties suppressed. The shear characteristics of the link were incorporated into an NL link element connecting two coincident nodes at the midpoint of the link. The shear properties of the NL Link were modelled as a tri-linear post yield curve. The initial shear stiffness was evaluated from shear properties and the shear link length. The expected strength was taken as 1.15 times the nominal strength. The peak strength of the second segment was taken as 1.35 times the expected strength. The rotations of links were limited to 0.3% at yield, 6.3% at peak strength and 9.3% as a maximum limit.

The Raleigh initial stiffness damping model available within SAP2000 was adopted. The damping assumed for the first mode was 5%, and a minimum of 2% damping was imposed for other modes.

Gravity loads from the tributary area for the two-dimensional frame were applied as uniformly distributed loads (UDL) on the beams, and the remaining load on the floor was applied in the P-Delta column, which was modelled with pinned ends as described in the Commentary to NZS1170.5. A dummy 'drift column' was modelled with unit shear stiffness so that the shear force within the column between two floor levels would give corresponding inter-storey drift, and the absolute maximum value out of max-min pairs of results was extracted..

Table 2 List of ground motions considered in this study

Shallow soil, Wellington					
Record Name	**Comp.**	**Station Name**	**Earthquake Name**	**M_W**	**Dist. (km)**
ARC2	N90E	Arcelik	1999 Kocaeli, Turkey	7.3	14
DUZ2	270	Duzce	1999 Kocaeli, Turkey	7.3	14
ELC2	270	El Centro	1940 El Centro	7.0	7
LAU1	S00E	La Union	1985 Michoacan	8.1	121
LUC1	260	Lucerne	1992 Landers	7.3	2
K0392	NS	HKD085	2003-09-26 Japan	8.3	45
TAB2	NS	Tabas	1978 Tabas, Iran	7.4	2
Deep soil, Wellington					
ELC1	180	El Centro	1940 El Centro	7.0	7
ELC2	270	El Centro	1940 El Centro	7.0	7
DUZ2	270	Duzce	1999 Kocaeli, Turkey	7.3	14
YPT1	60	Yarimca	1999 Kocaeli, Turkey	7.5	5
YPT2	330	Yarimca	1999 Kocaeli, Turkey	7.5	5
K0391	EW	HKD085	2003-09-26 Japan	8.3	45
K0392	NS	HKD085	2003-09-26 Japan	8.3	45
Shallow soil, Auckland					
A-BEN1	270	BENTON	1986 Chalfant Valley	6.2	14
A-LAD1	180	Bishop - LADWP	1986 Chalfant Valley	6.2	24
A-LAD2	270	Bishop - LADWP	1986 Chalfant Valley	6.2	24
BRA1	225	Brawley Airport	1981 Westmorland	5.9	15
PTS1	225	Parachute Facility	1981 Westmorland	5.9	17
PTS2	315	Parachute Facility	1981 Westmorland	5.9	17
G031	NS	Gilroy Array #3	1979 Coyote Lake	5.7	7

5 GROUND MOTIONS

Ground motion records with seismic signatures for shallow and deep soil sites for Wellington and only shallow sites for Auckland were chosen from the GNS library of ground motion records (Table 2).

The records were chosen such that when scaled using the scaling procedure given in NZS1170.5, they matched the target spectrum over the spectral range specified in NZS 1170.5 (0.4 T_1 <T_1<1.3T_1). The target spectra used were the design spectrum with 500-year return period for the chosen locations. The last number in the record name refers to the component number. Intensities of all the 7 ground motion components chosen were modified using appropriate scale factors to match the target spectra.

6 INTERSTOREY DRIFTS

For all of the buildings, inter-storey drifts from MRA and ITHA methods were compiled and presented in this section. It should be noted that the 'actual' responses from the analyses were considered for comparison and not reduced by any scale factor even if the records included forward directivity [NZS 1170.5 Clause7.3.1.2]. Due to space restrictions not all the results are presented here but available elsewhere [Uma et al., 2009].

6.1 Wellington low-rise buildings

For RC frame buildings, the ITHA derived inter-storey drifts were less than those obtained from the MRA approach at lower storeys, but exceeded the MRA results in the upper storeys. In both ductile (RC3WCD) and limited ductile (RC3WCL) buildings, LUC1 and ARC2 records resulted in higher inter-storey drifts (ISD) values compared to other records, for which the possible reasons are discussed in Section 7. MRA results for both buildings included drifts from P-delta forces. The limited ductile building was flexible enough to experience slightly increased drifts under MRA whereas the inelastic drift demands were almost the same.

For steel buildings, the inelastic drift demand at the 1st storey level was greater than those in other storeys. In the case of limited ductile building(ST3WCL), the section sizes were required to be stiffer to meet the higher shear demand, and P-delta forces did not contribute towards computed MRA results (NZS 1170.5 Clause 6.5.4.2). This obviously resulted in slightly lower MRA values than those observed for ST3WCD. The inelastic demands for both buildings were closer indicating that design compliance with the code and the maximum member capacities were realised. However, the maximum ISD values were much less than the code limit of 2.5% in all cases. Only the responses of ductile buildings response are shown in Figure 3.

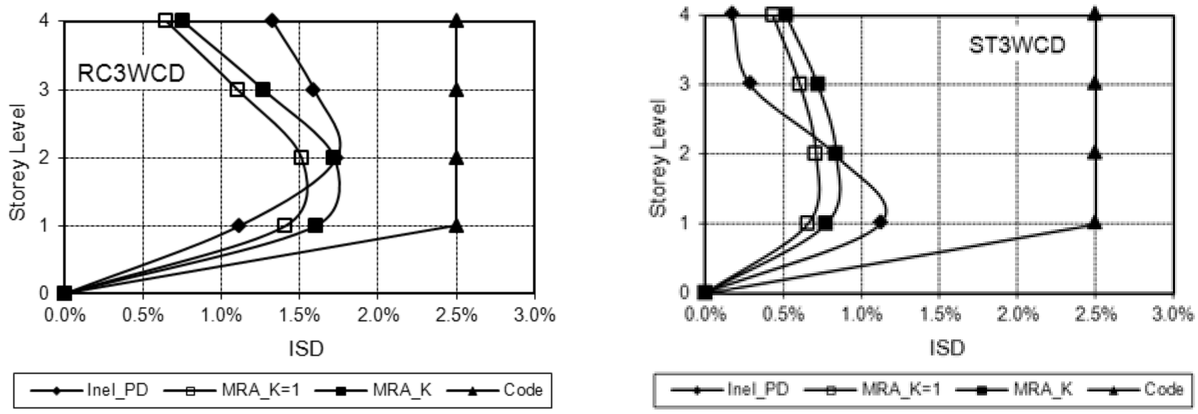


Figure 3 Inter-storey drift profiles for Wellington low-rise buildings

6.2 Auckland low-rise buildings

For Auckland region, RC and steel frames were designed with limited ductility of 3. Note that the buildings, particularly the steel buildings, could realise drifts much less than the 2.5% target drift limit. One of the reasons is that the stress limits in the 'links' and other design checks controlled the design resulting in stiffer frames. For RC building, it can be observed that the ISD profile is less than the MRA derived values. For steel buildings, inelastic ISD values exceeded MRA values only for the 1st storey (Figure 4).

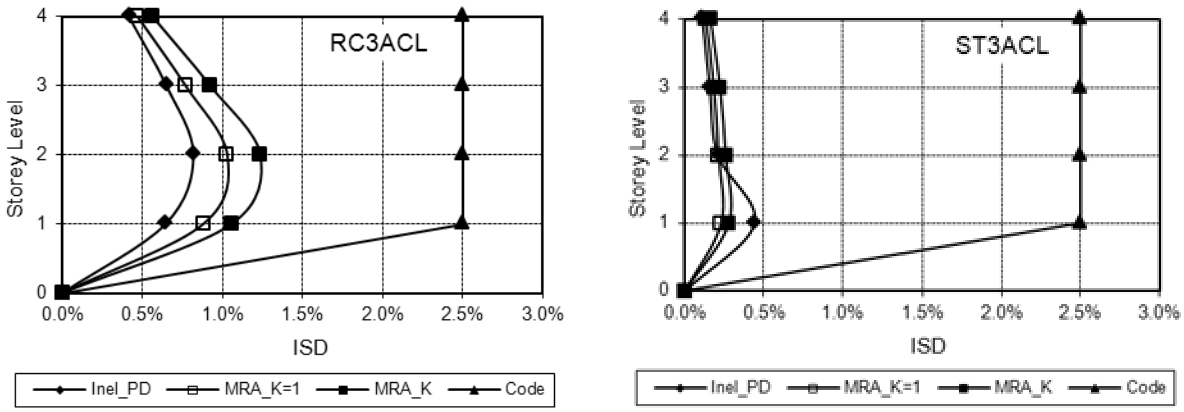


Figure 4 Inter-storey drift profiles for Auckland low-rise buildings

6.3 **Wellington high-rise buildings**

In RC frame buildings, inelastic ISD values slightly exceeded MRA values at the lowest and upper floors only (Figure 5). It can be noted that the MRA curve with drift modification factor, k_{dm} is around 2.25% which can be considered to be approaching the code limit. Figure 5 shows that the inelastic ISD is very close to the MRA_K=1 curve in lower and upper storeys and much less in the middle 1/3 height of the building. Hence, the justification for inter-storey drift multiplier by $k_{dm} \neq 1.0$ remains doubtful.

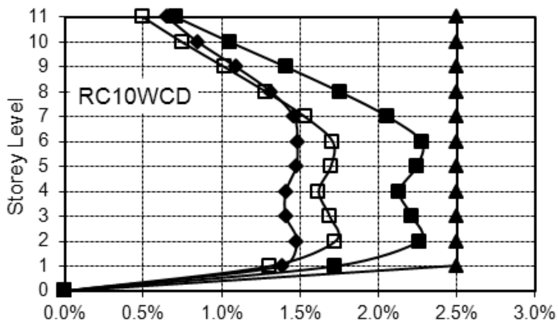


Figure 5. Inter-storey drift profiles for Wellington high-rise RC building

The inelastic ISD profiles for ductile and limited ductile buildings are shown in Figure 6. It may be noted that the limited ductile building shows higher ISD demand localised at the lower storey level. The change in sections up the height of the building resulted in the change in ISD demand at those levels. For example, in ST10WCD building, the stiffest section was at the 1st storey level; member sizes were changed at the 2nd storey, 5th storey and 8th storey.

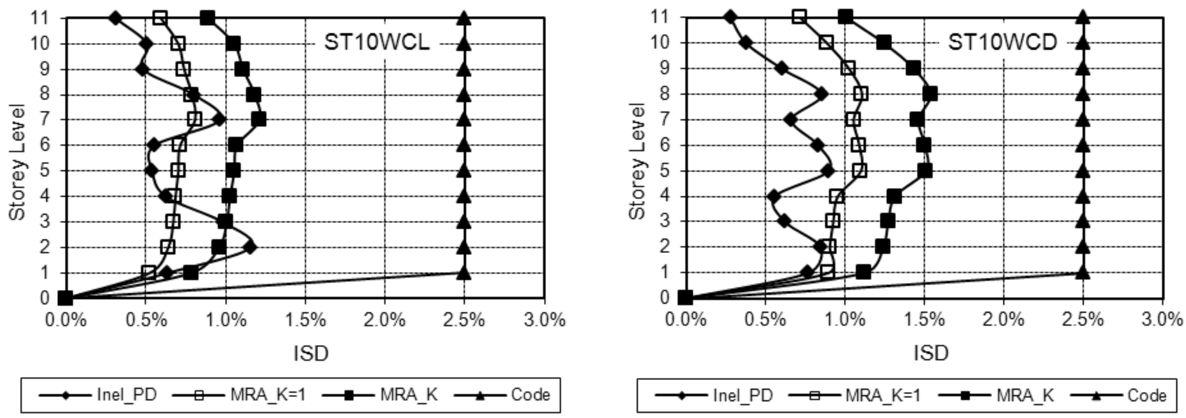


Figure 6 Inter-storey drift profiles for Wellington high-rise steel building

For the ST10WCL building, a very stiff section was used at the 1st storey level. The reduction in member size at the 2nd storey level was large, hence resulting in quite a high demand in ISD at that level. Further change in member sizes were adopted at 7th storey and 10th storey level which explains the ISD profile pattern as shown in Figure 6. Note that the ARC2 record gave extreme results for both ST10WCD and ST10WCL, and hence the responses it generated were considered outliers and were disregarded.

6.4 Auckland high-rise buildings

The ISD profile (Figure 7) shows 50% increase in demand compared to MRA_K=1 at the lowest storey level. However, it should be noted that the maximum ISD that was realised was only about 0.35%, which is one order less than the code limit value. The ISD profile is somewhat lower than MRA profile at all levels (Figure 7). Similarly to low-rise steel buildings, the design checks with regard to stress limits in the elements resulted in a stiffer frame; hence the reduced drifts compared to the target drift value.

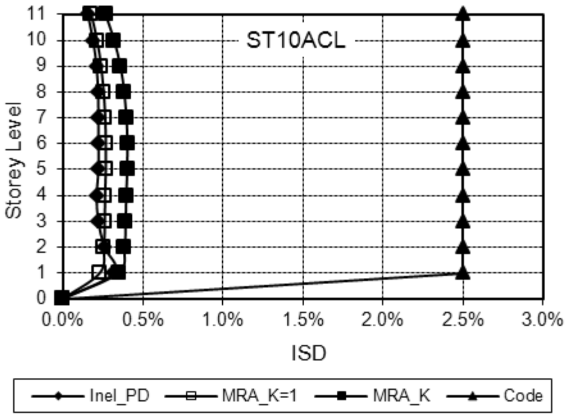


Figure 7 Inter-storey drift profiles for Auckland high-rise steel buildings

7 EXTREME VALUES FROM CERTAIN RECORDS

During the ITHA on buildings considered in this study, a few records resulted in very high responses or instability of the structure. Such extreme cases were not included in deriving the scale factors. The buildings and the corresponding records that showed such anomalies are listed in Table 3.

Table 3 List of buildings and records that resulted in extreme responses

Sl No	Building	Records eliminated
1	RC3WCD	K0392
2	ST3WCD	LUC1
3	ST10WCD	ARC2
4	ST10WCL	ARC2
5	ST10WDD	YPT1

The NZS1170.5 procedure was followed to select the records and to arrive at the scale factors to match the target spectrum. In spite of this, the responses from a few records were very high compared to the values from the rest of the records considered in the suite. This observation was examined for all the buildings listed in Table 3 and discussed only with reference to RC3WCD building.

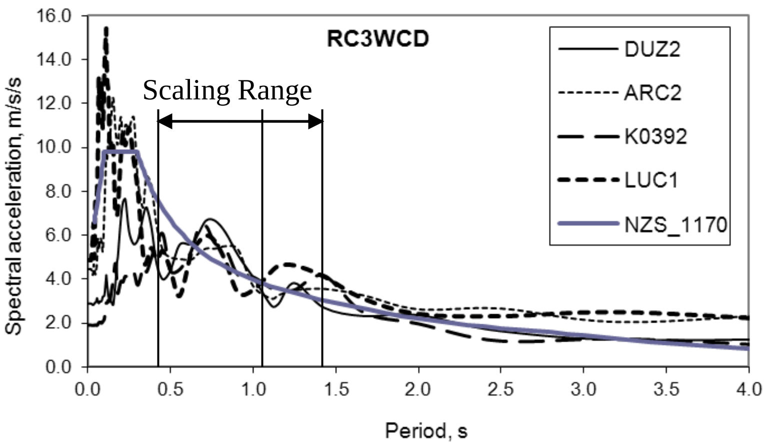


Figure 8 Comparison of response spectra of select records with design spectrum of NZS_1170

The fundamental period of the building was 1.1s. The period range considered for matching was between 0.44s and 1.4s. Out of the 7 records considered, the 'K0392' record encountered convergence failure. Also, records LUC1 and ARC2 resulted in large ISD values. It is clear from Figure 8 that the records LUC1, ARC2 and K0392 exceeded the target spectrum at periods beyond the scaling range compared to DUZ2. Note that the effective period of building under inelastic response is generally gets increased compared to the initial period. Hence the records which exceeded the target spectrum in the extended period range could have resulted either in higher responses or in convergence failure during ITHA. Also, the previous cyclic history of the building for these records could be a possible influencing parameter for extreme responses for those records.

8 SCALE FACTORS FROM THE PRESENT STUDY

The scale factor is derived as the ratio of ISD from ITHA to that from MRA procedures without any drift modification factor, as denoted by (MRA_K=1) and plotted for all the buildings considered within this study as shown in Figure 9.

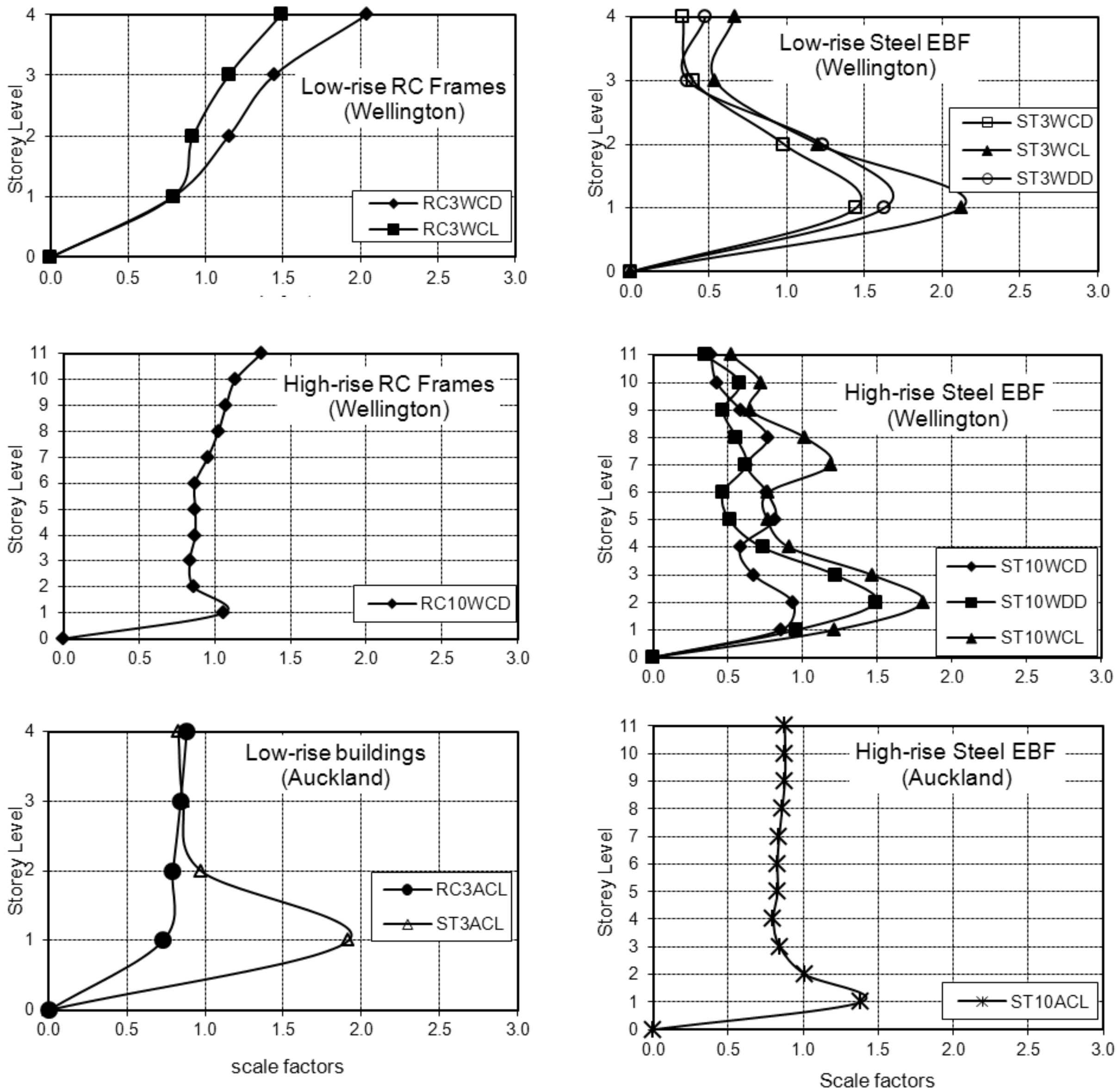


Figure 9 Scale factors obtained for the suite of buildings considered in this study

For Wellington low-rise concrete buildings, even though the scale factor for limited ductile building is higher than its ductile counterpart, the ISD were not very different. The difference in the scale factor is due to the difference in MRA results from P-Delta effects.

The current provisions in NZS1170.5 recommend a drift modification factor, k_{dm} to increase inter-storey drifts at every storey level determined by MRA procedures. The present study shows that: (i) the amplification (i.e. representative scale factors) is not constant at every storey level up the height; (ii) in general, the drifts at lower storeys are amplified more than those at upper storey levels (with the exception of low-rise concrete buildings); and (iii) at some storey levels the ‘actual’ drifts are less than those determined by MRA (i.e. scale factors < 1.0).

An attempt is made to summarise the scale factors based on the limited analytical results from this present study, and to recommend a new set of drift modification factors for low-rise (~15 m) high-rise (>30 m) buildings (as listed in Table 4).

Table 4 Recommendations for revised drift modification factors

Type of structure	Location	NZS_1170.5 Drift modification factor	Recommended Drift modification factor
Low-rise RC Frames	Wellington	1.2	1.25 at each storey level
Low-rise RC Frames	Auckland	1.2	1.0 at each storey level
Low-rise steel EBF	Wellington	1.2	1.5 in lower two storeys and 1.0 elsewhere
Low-rise steel EBF	Auckland	1.2	1.5 in lower two storeys and 1.0 elsewhere
High-rise RC Frame	Wellington	1.5	1.25 at each storey level
High-rise steel EBF	Wellington	1.5	1.5 in lower 1/3rd height and 1.0 elsewhere
High-rise steel EBF	Auckland	1.5	1.0 at each storey level

In the opinion of the authors, the scale factors for medium-rise buildings shall be obtained by interpolation as suggested by NZS 1170.5.

9 SUMMARY AND CONCLUSIONS

In this study the current provisions related to drift modification factors in the loading standards NZS 1170.5 were verified by analysing different structural systems located in different seismicity regions by elastic and inelastic procedures. From the limited number of buildings analysed, it was noted that the ISD values from both ITHA and MRA were within the prescribed code limit of 2.5%. Further, it was observed that the ratio of inter-storey drifts between ITHA and MRA was not constant up the height of the building, whereas the recommended practice is to adopt a single value up the height. The drift modification factor is influenced not only by the height of the building but also by the structural system adopted and the seismicity level. Revised drift modification factors are suggested for different structural systems located in high and low seismicity regions.

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