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Acceptable inter-storey drift limits for buildings at ultimate limit states

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Acceptable Inter-Storey Drift Limits for Buildings at Ultimate Limit States

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NOTATION

A_g	=	cross sectional area of the gross concrete section (mm^2)
A_s	=	cross sectional area of reinforcing steel (mm^2)
d	=	effective depth of reinforcing tension steel (mm)
d'	=	effective depth of reinforcing compression steel (mm)
f'_c	=	characteristic strength of concrete (MPa)
E_c	=	modulus of elasticity of concrete (MPa)
E_s	=	modulus of elasticity of steel (MPa)
f_y	=	yield strength of reinforcing steel (MPa)
I_e	=	effective moment of inertia (mm^4)
Ig	=	effective moment of inertia of the gross concrete section (mm^4)
L	=	Length
L_p	=	plastic hinge length (m)
L_w	=	length of wall (m)
M_n	=	nominal moment capacity (kNm)
M_o	=	overstrength moment capacity (kNm)
ϕ	=	strength reduction factor, as defined in NZS 3101
μ	=	structural ductility factor
λ_e	=	expected strength factor
λ_o	=	overstrength factor

EXECUTIVE SUMMARY

This project was initiated to validate (or modify) the displacement modification factor, k_{dm}, published in Clause 7.1 of NZS1170.5 Earthquake Design Provisions for New Zealand 2004. Those provisions stemmed from research undertaken prior to the publication of NZS 4203:1992 within which it was noted that displacements, interstorey displacements in particular, derived using elastic analysis techniques and scaled for Ultimate Limit State conditions, considerably underestimated the equivalent displacements derived from dynamic analysis.

This study has involved the development of fifteen 2-D structural forms that were compliant with the interstorey drift criteria stipulated in the loading standard and their respective material standard. The structures were then analysed using Inelastic Time History Analysis (ITHA) techniques with selected earthquake records that complied with the provisions of 1170.5. Comparisons were then made with the interstorey drifts calculated for each structure between the displacements assessed from each method, i.e. the scaled elastic response (in all cases Modal Response Analysis (MRA) was engaged for that phase) and those determined from the ITHA.

The selection of the building form used in the study involved two seismic settings (High – Wellington and Low – Auckland), two site classes (Class C and D each of which was assigned a suite of 7 compliant ground motion records), three structural forms (Reinforced concrete frame : μ=6 , RC Wall: μ=5 and Eccentrically Braced Steel Frames: μ=4) and three storey heights (3, 10 and 20 storey plus light roof above each upper floor).

The key findings from the study are:

- Only in the case of RC frames did the scaled elastic displacement profile provide a reasonable match with the ITHA derived profile. The ability to predict post-elastic deformation by scaling the elastic profile in all other cases is therefore highly doubtful.
- For RC frames, k_{dm} = 1.25 would appear appropriate.
- For RC walls, nearly all post-elastic deformation occurs within the plastic hinge zone at the base of the wall, above which the interstorey drifts are uniform (ie the building responds as a rigid body with base rotation). Interstorey displacements are therefore only an issue over the plastic hinge zone (perhaps 2x wall length) and a k_{dm} = 1.5 is recommended over the plastic hinge zone for buildings of this form with k_{dm} = 1.0 elsewhere.
- For EBF steel buildings, the in-elastic displacement is limited to the lower 1/3 of the building height, above which the building responded as a rigid body, maintaining its original verticality. For the lower 1/3 of the building height of such structures, k_{dm} = 1.5 would appear appropriate with k_{dm}=1.0 above that height.
- In many cases the minimum dimensional and sectional steel ratios not only dictated strength but also controlled both the post-elastic ductility and the interstorey drifts experienced. In such cases within this study, the displacements were well below (50%) of the drift limits set by 1170.4. This was particularly the case for the Auckland buildings

where, although k_{dm} ranged between 1.4 and 1.9, the absolute values in all cases were well below the control criteria. The above recommendations of k_{dm} are still therefore considered to be appropriate while the minium reinforcing ratios in the existing material standards continue to be applied. Should these change in the future, care will be required to ensure soft-storey effects are avoided within the lower storeys.

This study was scoped to equate to approximately $100k half of which was sought and provided by EQC and the remainder being unsuccessfully sought from other commercial sponsors. In the event, the actual expenditure amounted to approximately 200K, with the contract period requiring to be extended accordingly. The implications from the study, while continuing to focus on k_{dm}, have also provided insight into the (conservative) nature of the existing drift limits and the influence minimum dimension requirements and steel ratios from the materials standard have on both the provision of adequate strength and ductility but also on compliance with displacement controls.

The conclusions reached in this study are based on the 15 building studied only. They have not considered torsion or 3-D dynamic response. While the number of buildings remains a minimum up which to draw recommendations, the research team are of the view that the results are robust and can be applied to other buildings of similar typology and those within the more moderate seismicity zones across New Zealand.

1.0 INTRODUCTION

Design engineer often use quasi-static procedures to analyse buildings and to determine the inter-storey drift demands. Building code compliance as stipulated in the new loadings standard NZS1170.5 (Standards New Zealand 2004) requires that these drifts, when scaled to simulated post-elastic deformations, be maintained within prescribed acceptable limits. The aim of the current research is to confirm the published drift scaling method or to develop rational alternative scaling procedures to justify compliance with the drift limits stated within that standard. It is also important that there is consistency between the ultimate limit state (ULS) drifts calculated using inelastic time history analyses (seldom used for design in NZ but considered to most accurately reflect the building behaviour) and those derived using elastic analysis techniques. Such elastic design procedures remain by far the most prevalent method of analysis with modal analysis now becoming the norm for buildings over 4 storeys 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.

Anomalies have been observed in the deformations assigned to buildings at their ultimate limit state depending on the procedure used to calculate the ULS displaced shape as suggested in NZS4203 (Standards New Zealand, 1992). The issues are further complicated by recognition that the deflections computed using equivalent static design could, depending on the ductility and number of stories, over-estimate deformations by up to 15%. A note of uncertainty was realised if the deformations were derived directly using inelastic time-history analysis or by scaling elastic deformations.

Hence, the aim of this research programme is to provide means of scaling inter-storey drifts derived using elastic analysis methods (equivalent static or modal) to align with the earthquake induced deformations limits published in NZS1170.5. The main steps undertaken in the research programme were as follows:

1. Design of a suite of buildings complying with NZS1170 and estimation of inter-storey drift responses using Modal Response Analyses (MRA): This task was undertaken by Dr. Darrin Bell and Tony Holden from Connell Wagner Wellington under contract to GNS Science.
2. Selection of a family of ground motion records using the criteria from NZS1170.5 suitably scaled to match the design spectrum.
3. Performance of inelastic time history analyses on each of the buildings using the simulated ground motion records from step 2.
4. Comparison of the results from inelastic and elastic methods of analyses and suggestion of suitable scale factors for design purposes.

Steps 2 to 4 were carried out within GNS Science.

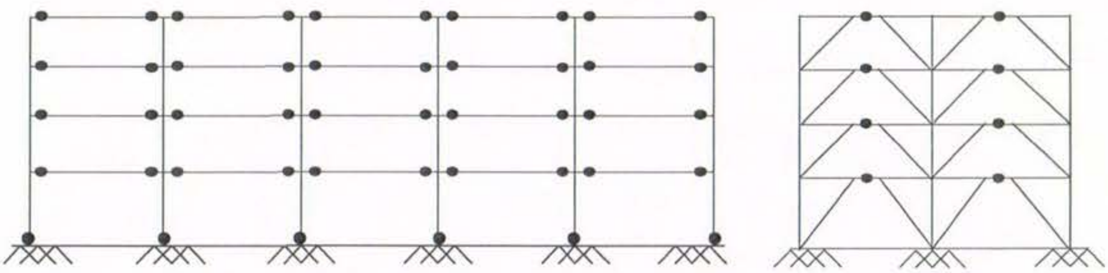
The work considered a suite of reinforced-concrete frame and wall structures and eccentrically-braced steel frames designed for locations with high and low seismicity and shallow and deep soil conditions. Elastic modal analyses following the procedure described in standard NZS1170.5 were performed to obtain inter-storey drifts. Through comparison of the results, the required scaling of elastic inter-storey drifts to the required drift limits is proposed.

2.0 BUILDING DESIGNS

2.1 Suit of buildings

Selection criteria for the buildings included the geometry (number of storeys), material of construction, site soil conditions, locations with respect to seismicity and ductility of the building. Basic design parameters including the applied building loading and configurations were chosen in line with a previous study (Shelton, 2004). Three building heights (3, 10 and 20 occupied floors), two material types (reinforced concrete (RC) and steel (ST)), two soil types (shallow and deep), two levels of seismicity (Wellington and Auckland) and two levels of ductility were specified. These parameters were used to develop the building models. Separate building models were developed for reinforced concrete Moment Resisting Frames (MRF) and Shear Walls (SW) and Eccentrically Braced Frames (EBF) steel buildings. Two examples of the model buildings are shown in Figure 1.

Two dimensional models were used for all cases. As a result, the effect of torsional irregularity was not addressed.



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

- Location where non-linearity is modelled

Figure 1: Typical 2-dimensional model for 3-storey frame with a roof for reinforced concrete frame and eccentrically braced steel frame.

The buildings considered in the study are summarised in Table 1, with the identification name being given in column 1.

Table 1. Building details

Identification Name	Structural Material	Number of storeys	Structural Form	Location	Soil Class	Ductility		Period (T1)
						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
RW10WCD	RC	10	Wall	Wellington	C	D	5	2.3
RW10WDD	RC	10	Wall	Wellington	D	D	5	1.8
RW20WCD	RC	20	Wall	Wellington	C	D	5	3.2
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

2.2 Design criteria

2.2.1 Basic design parameters

Two dimensional models were developed from the building configurations that were adopted in an earlier study conducted by BRANZ (Shelton, 2004). Some modifications were made to the previously assumed seismic mass of the building. For example, the seismic mass was increased by 25% to ensure drifts were approaching code limits.

2.2.2 Building configuration

Reinforced concrete buildings were configured with two seismic frames in the longitudinal direction and two shear walls in the transverse direction. The building responses were studied in two orthogonal directions independently. Hence, separate two-dimensional models were created for a seismic frame and a shear wall to represent reinforced concrete frame buildings and shear wall buildings. In steel buildings, there were two seismic frames in one direction and two eccentrically braced frames (EBF) in the other direction. Only the EBF frames were modelled in this study.

The guidelines adopted in the building schemes were:

1. The overall layout of building plans were as same as these considered in the previous study (Shelton, 2004). Buildings were considered to be regular. Seismic frames and walls of interest were designed for the tributary loading.

2. 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 with a span of 7.5 m distance. Buildings with shear walls utilised either one or two walls for a bay width of 9.0 m depending on the design requirement. EBF frames were modelled with two bays at 8.5 m distance.

3. A roof was included in each building above the upper occupied floor resulting in the additional level above the specified number of storeys.

4. The building mass was assumed to be uniformly distributed over the entire floor area, and the seismic mass of each floor was derived from the contributing mass from the tributary area. The mass was lumped at a single node on each floor, and all such masses were aligned vertically along a common reference line. The seismic masses for the structure (reinforced concrete frame, shear wall or EBF) were 548 t per floor, except for the top floor which was 515 t.

5. Structural members were sized so that as far as practicable the design strength was the minimum code compliant level for the specified level of ductility. For the reinforced concrete structures, the minimum steel provisions prescribed by NZS3101 (Standards New Zealand, 2006) were followed.

6. Beams varied in size over the height of the buildings to assist in matching design strength with demand. However, only 3 to 4 changes of beam sizes were adopted to reflect typical building practice.

2.2.3 Design aspects

The member sections were so chosen as to provide enough flexibility to the building to enable the inter-storey drift ratios from Modal Response Spectrum Analysis (MRA) to reach the code limit of 2.5%. Figures 2 and 3 show the inter-storey drift profiles obtained from MRA with Drift Modification Factors (DMF) (as per NZS1170.5 clause: 7.3.1.1) of k_{dm}=1 and k_{dm}=1.2. It can be observed that the maximum inter-storey drift (ISD) was around 1.5% for the RC3WCD building, which is well below the code limit, and was close to 1.9% for RC3WCL building.

When considering Figures 2 and 3 note that MRA_K=1 refers to the drift results with no DMF (i.e. k_{dm}=1.0), and MRA_K refers to the drift results with DMF appropriate for the type of building. This applies to many of the subsequent figures.

For the steel buildings, the maximum ISDs from MRA were much less than 2.5% (Figures 4 and 5). Attempts made to increase the ISD to reach the code limit of 2.5% were not successful for the following reasons. While ductile behaviour has to be confined to ends of the beam members for RC buildings and to the active links for steel EBF buildings, the respective material standards specify other requirements be satisfied so as to achieve ductile behaviour in the members. For example, in case of RC beams, the dimension limitation of beam sections comes from of the clause 9.4.1.2 and minimum steel requirement comes from clause 9.4.3.4 of NZS 3101:2006. When the additional requirements control firstly section dimensions and then strength, the target drift limits are often well in excess of those experienced by the model. Therefore, it should be recognised that the limit state for the section dimension was not governed by drift limits, but from other specified criteria. In the case of steel EBF building, the size of beam, brace and links were so chosen that stresses within the links were less than the allowable limit. In these cases, other practical requirements from the respective material standards resulted in the ISD of the designed buildings being much below the maximum code drift limit of 2.5%. Similar reasons can be attributed to other buildings as well.

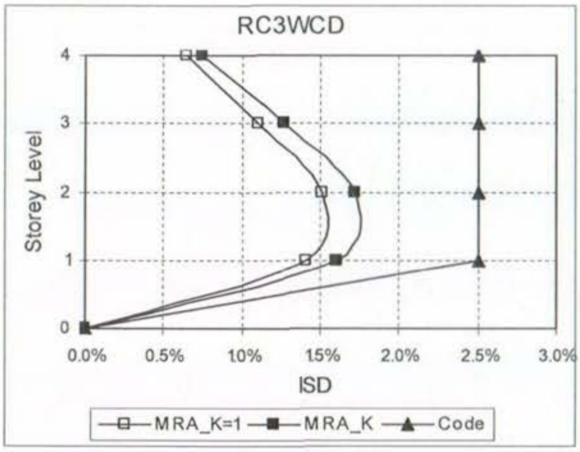


Figure 2: MRA results for RC3WCD

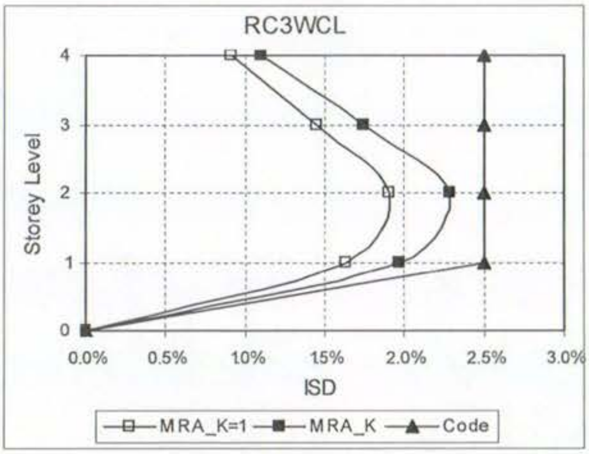


Figure 3: MRA results for RC3WCL

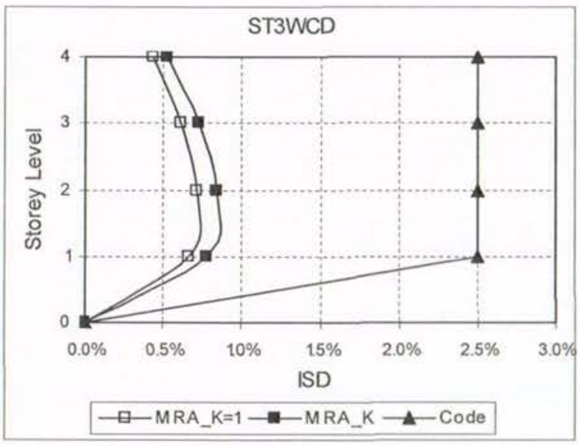


Figure 4: MRA results for ST3WCD

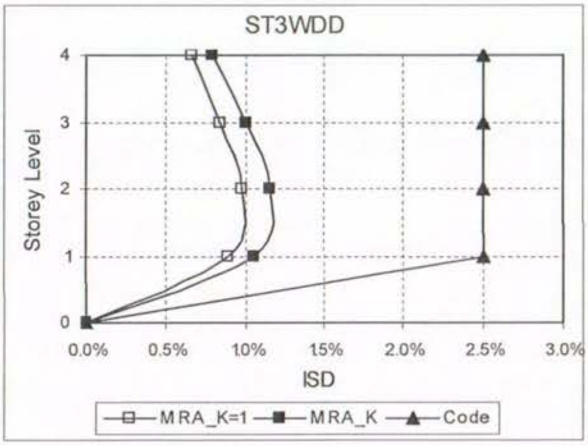


Figure 5: MRA results for ST3WCL

2.2.4 Modelling assumptions and methods

Procedures as described by NZS1170.0 were adopted to design the buildings with appropriate loads and load combinations. The ETABS finite element analysis program was used for the design. Care has been taken to proportion the member sizes so that plastic hinges occurred in columns only at the base level, elsewhere column hinging was suppressed with all hinging being confined to the ends of beam member.

Reinforced concrete buildings:

1. Concrete members were designed and detailed in accordance with the Concrete Structures Standard, NZS 3101 (Standards New Zealand, 2006)
2. Beam design strengths were calculated based on $\phi M_n = \phi A_s f_y (d - d')$. The slab mesh, along with any additional longitudinal bars required for stirrups, was neglected in calculations for design strength.
3. Beam maximum over-strength bending moment capacity was taken as $\lambda_o / \phi M^*$ (where M^* is given as the greater of redistributed bending moment or based on minimum rebar content for the section; λ_o = 1.25 and 1.35 for f_y=300 MPa and f_y=500 MPa respectively; ϕ = 0.85). It was assumed that no slab steel was present to contribute to over-strength.
4. Rebar expected strength was taken as $1.15 f_y$, i.e. 1.15 times the lower characteristic strength
5. Concrete expected strength was taken as 1.5 times the specified concrete strength, f_c'
6. Concrete expected stiffness was taken as $E_{ce} = 1.3 E_c$ where E_c is the dependable Youngs modulus derived from f_c'.
7. Reinforcement was curtailed to ensure that plastic hinging occurred at the column face.
8. Plastic hinge length on beams L_h was taken to be 0.67 times the beam depth. Linear effective stiffness was set as $E_{ce} I_e / L_h$, in which I_e was effective moment of inertia of beam's cross section.
9. Plastic hinge on walls was considered as $L_h = 0.2 L_w + 0.07 M/V$ ($\approx$3.6m for 9m wall)

Structural steel buildings:

1. Steel members were designed and detailed in accordance with the Steel Structures Standard NZS3404 (Standards New Zealand, 1997)
2. For steel links, the expected strength was taken as 1.15 times the nominal strength and the peak strength was taken as 1.35 times the expected strength.

2.2.5 Analytical models

The analytical models were prepared in two different software platforms. ETABS was used to design the buildings to arrive at the member sizes and to obtain results from Modal Response Analyses. SAP2000 Version 11 and V12 were used to carry out inelastic time history (ITHA) analyses.

The floor was considered to be a rigid diaphragm in all the buildings considered in this study. A P-Delta column was modelled to lump the P-Delta weight of the structure. The procedure described in NZS1170. 5 as in Method B was adopted to model the P-Delta column which assumes pin ends (as moment releases) at the bases of the columns at every storey level.

Modelling of reinforced concrete buildings:

1. Beams and columns were modelled using frame elements (SAP2000). The frame element used a general, 3-dimensional beam-column formulation, which included the effect of biaxial bending, torsion, axial deformation and biaxial deformation (Bathe and Wilson, 1976).
2. Beam-column joints were represented by the rigid end offsets of the respective beams and columns. The rigid end-offsets were taken as half the depth of the member.
3. Shear wall was considered as columns having the equivalent stiffness and strength of the walls.
4. The base of the structure was assumed to be fixed.

Modelling of structural steel buildings:

1. Beams, columns and braces were modelled using frame elements
2. End offsets for beams and columns were not considered and hence the panel zone flexibility was not included.
3. EBF frames were modelled with fixed base support for time history analyses but were assumed as pinned conditions for design analysis.

Seismic loads:

Seismic loading was evaluated in accordance with the loadings standards (AS/NZS, 2004). Further notes are as follows:

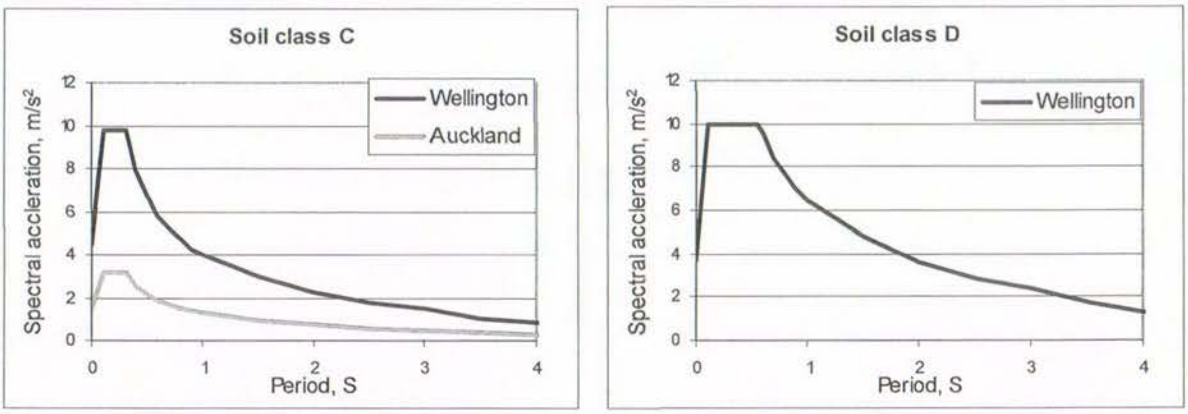
1. A structural performance factor, Sp, factor of 0.7 was used for all buildings
2. Return period factor, R was taken as 1.0 for all the buildings
3. Minimal seismic coefficient was assumed
4. Base shear scaling for response spectrum analyses was done

2.3 Response spectrum analyses

The drifts from the elastic method of analysis were obtained using the "Response spectrum analysis" option in ETABS. For buildings at Wellington sites, the distance of the fault was considered to be less than 2 km. The modal results were combined using the Complete Quadratic Combination method (CQC). The drifts from P-Delta effects were added to the modal results following the Method B procedure described in NZS1170.5. The results were obtained with and without the drift modification factor, k_{dm} and are presented in Appendix A. The first mode periods of the buildings analysed (T_1) are given in Table 1 above.

3.0 GROUND MOTIONS

Records were selected from the GNS library of ground motion records. Three sets of ground motions (each with 7 components) with seismic signatures for shallow soil sites (Wellington and Auckland) and deep soil sites (Wellington) (giving a total of 21 components) were chosen. The records were chosen such that when scaled using the scaling procedure as 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. Target Spectra for Wellington and Auckland are given in Figure 6.



(a) Shallow soil (b) Deep soil

Figure 6: Target spectra from NZS1170.5

3.1 Suite of ground motions

The details of the chosen records with horizontal components are given in Table 2. The last number in the record name refers to the component number.

Table 2: List of ground motions considered in this study

Record Name	Comp.	Station Name	Earthquake Name	M_W	Dist. (km)
Shallow soil, Wellington					
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	Lucern	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 South St	1986 Chalfant Valley	6.2	24
A-LAD2	270	Bishop - LADWP South St	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

4.0 TIME HISTORY ANALYSES

4.1 Over view

The exercise for the integration time-history analyses (ITHA) with SAP2000, a commercially available computational package (Computers and Structures, Inc., Berkeley CA), helped to study the features and computational efficiency of the software package in performing integration time-history analyses, apart from the convenience with the pre-processor to prepare input model and the post-processor to retrieve the results. The features that were available in the software and used for this research were:

1. Modal Analysis to find the mode shapes and periods
2. Response spectrum analysis (for any target spectrum)
3. Non-linear static analysis
4. P-Delta option
5. Nonlinear integration time-history analysis
6. S_p factor of 0.85 is used as per NZS 1170.5 for time history analyses

7. Hysteretic models (Takeda model and multi-linear kinematic model were used to simulate the inelastic cyclic behaviour of reinforced concrete elements and for steel links respectively)

Two dimensional models for all building types were prepared using the graphic interface. The mode shapes and the associated periods of all buildings were obtained using the Eigenmode analysis option. The scale factors for all the chosen records were obtained to match the target spectra within the period range as suggested in NZS1170.5. Intensities of all the 7 ground motion components chosen were modified using appropriate scale factors to match the target spectra. The analysis time-step was determined by the software to meet the convergence criteria at every time interval. The scale factors corresponding to the records chosen are listed in Table 3 (a), (b) and (c) for two soil sites and two locations considered.

Table 3: Scale factors used to match the target spectrum

(a) Shallow soil, Wellington Buildings

Records	Period of buildings (s)					
	0.7	1.1	1.5	1.75	2.3	3.2
ARC2	3.5	3.65	3.55	3.55	3.5	3.09
DUZ2	0.75	0.65	0.66	0.6	0.76	0.79
ELC2	1.1	1.1	1.36	1.34	1.56	1.95
LAU1	1.98	1.94	2.15	2.2	2.4	2.85
LUC1	1.06	0.93	0.83	0.82	0.81	0.71
K0392	1.02	0.81	0.78	0.83	1.05	1.13
TAB2	0.56	0.53	0.55	0.56	0.64	0.7

(b) Deep Soil Wellington

Records	Period of buildings (s)		
	0.7	1.6	1.8
DUZ2	1.1	1.1	1.1
ELC1	2.26	2.6	2.67
ELC2	1.56	2.18	2.3
YPT1	1.68	1.9	1.9
YPT2	2.02	1.54	1.54
K0391	1.84	1.41	1.41
K0392	1.54	1.36	1.36

(c) Shallow soil, Auckland

Records	Period of buildings (s)		
	1.0	2.0	2.15
A-BEN1	0.89	0.87	0.91
A-LAD1	0.64	0.92	0.89
A-LAD2	0.80	0.97	0.96
BRA1	1.23	1.23	1.24
PTS1	0.55	0.45	0.43
PTS2	0.51	0.55	0.56
G031	1.11	1.21	1.22

Integrated time-history analysis (ITHA) requires input of the gravity load actions present in the structure at the start of the analysis. This was done by performing non-linear static analysis with gravity loads. The resulting actions provided the initial conditions for ITHA analysis.

4.2 Modelling aspects

The models used for ITHA were provided with material properties based on expected strength which is higher than dependable or specified strength. The structural models included nonlinear elements to reflect inelastic deformations.

4.2.1 Material modelling

The expected strength of concrete, $\lambda_{ec}\left(f_c^{'}\right)$, was assumed to be 50% higher than that specified. The concrete stiffness used for design was increased by 30% to get the expected stiffness of the concrete, $\lambda_e\left(E_c\right)$. Reinforcing steel expected yield strengths, f_{ye}, were taken as being 15% higher than specified, as per standard practice.

For steel buildings, the expected yield strength, $\lambda_{es}\left(f_y\right)$, was taken as 1.15 times the specified strength.

In summary:

- The factors used to calculate expected material strengths and stiffnesses for reinforced concrete buildings were based on:

  $\lambda_{ec}\left(f_c^{'}\right)$ = 1.50

  $\lambda_{es}\left(f_y\right)$ = 1.15

  $\lambda_e\left(E_c\right)$ = 1.30

- The factors used to calculate expected material strengths for the steel buildings

$\lambda_{es}(f_y)$ = 1.15

$\lambda_e(E_s)$ = 1.0

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.

4.2.3 Structural modelling

The structural models prepared for modal response spectrum analyses were modified to include elements that were capable of simulating inelastic deformation under reversed cyclic loads.

4.2.3.1 Reinforced concrete buildings

For concrete buildings, the member stiffness properties were modified to account for cracking. The effective stiffnesses, I_e provided for frame members are listed in Table 4.

Table 4: Effective stiffness for reinforced concrete members

Members	Beams	Internal columns	External columns	Walls
Effective stiffness	$I_e = 0.33 I_g$	$I_e = 0.5 I_g$	$I_e = 0.4 I_g$	$I_e = 0.33 I_g$

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. The effective stiffness of nonlinear link elements was expressed as $\left(E_{ce}\, I_e / L_p\right)$.

Assumed lengths of plastic hinge zones $\left(L_p\right)$ are:

Frames: 0.67 times the member depth

Walls: $0.2 L_w + 0.07(M/V)$

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 level of reinforced concrete frames were modelled as fibre hinges. However, the computational time for every run was increased greatly due to the use of NL link elements and fibre hinges. The batch file mode of operation was very useful to make the runs over night.

However, for wall buildings, since the variation of axial loads will not be very significant (when vertical motion was not modelled), the plastic hinge at the base of the wall was modelled with an NL link element.

4.2.3.2 *Steel buildings*

In eccentrically-braced frames, the column, brace and beam members were modelled using frame elements as discussed above. 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 (as AG/L). 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 of the expected strength. The rotations of links were limited to 0.3% at yield, 6.3% at peak strength and 9.3% as maximum limit. Figure 7 shows the backbone envelope of strength-strain ratio adopted for the steel links.

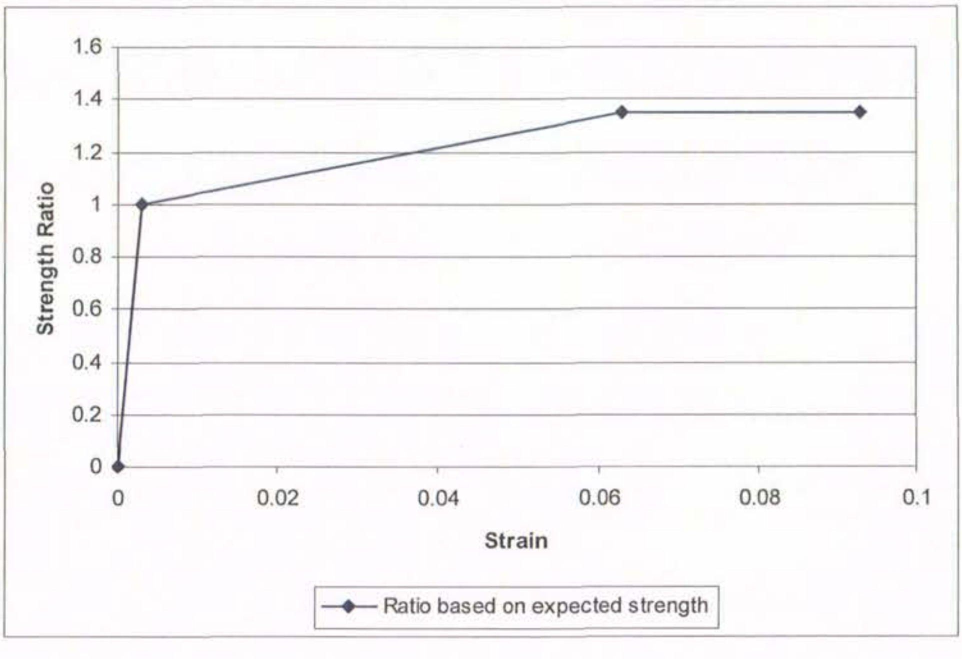


Figure 7: Strength-strain ratio assumed for steel links

4.2.4 Damping

The Raleigh initial stiffness damping model available within SAP2000 was adopted. The mass and stiffness proportional coefficients were computed within the software based on the first and second mode periods of the building. The damping assumed for the first mode was 5% and a minimum of 2% damping was imposed for other modes.

4.2.5 Loads

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.

4.3 Results extraction

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 pair of results was extracted.

The maximum displacement of the nodes on the dummy drift column was recorded to give the displacement profile of the structure.

5.0 RESULTS

For all of the buildings, inter-storey drifts from the elastic method (MRA) and the time-history method (ITHA) 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 Cl 7.3.1.2].

5.1 General observations:

1. Inelastic inter-storey drifts for steel buildings were less compared to those in the RC buildings.
2. For 3-storey buildings, larger linterstorey Drifts were experienced in lower storeys (1st storey level) for steel buildings but in the 2nd and 3rd storeys for reinforced concrete buildings. In all cases these remained within acceptable limits.
3. The ISD profiles for RC buildings from ITHA analyses aligned moderately well with MRA results, except that the drifts exceeded the MRA values for upper storeys.
4. The ISD profiles in steel buildings well exceeded the MRA values at lower storeys but not in upper storeys. In all cases being well below the 2.5% drift limit.
5. ISD values obtained from ITHA for RC ductile building RC3WCD and limited ductile building RC3WCL were similar. However, the MRA drifts were larger for the limited ductile building compared to the ductile building.
6. Displacement profiles for ITHA derived RC ductile and low ductile buildings were close to elastic profiles.
7. Displacement profiles for ITHA derived steel buildings invariably exceeded the elastic profiles, for lower storeys.
8. ISDs profiles for ductile and limited ductile steel buildings were almost the same in both patterns and values.
9. The ISDs at lower storeys of the steel ductile building (ST3WDD) located in deep soil were higher than those exhibited by ST3WCD located in shallow soil.
10. For Auckland site, RC buildings the ITHA derived displacements and ISDs were much less than MRA results over the heights of the buildings except for steel building, ISD was increased to double of MRA values. The inelastic displacement profile for ST3ACL

exceeded the MRA profile.

11. Even though the ISD values from ITHA were more than those from MRA, all values were within the prescribed code limit of 2.5%

5.2 Inter-storey drifts for 3-storey buildings

5.2.1 Wellington RC buildings (ductile vs limited ductile)

The ITHA derived inter-storey drifts for RC frames in Wellington shallow soil conditions were less than those obtained from the MRA approach at lower storeys, but exceeded the MRA results in the upper storeys, as shown in Figures 8 and 9. In both ductile (RC3WCD) and limited ductile (RC3WCL) buildings, higher values of ISD were identified with LUC1 and ARC2 records. The limited ductile building was more flexible than the ductile building and the fundamental period was slightly larger (1.1s of 1.5s). 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.

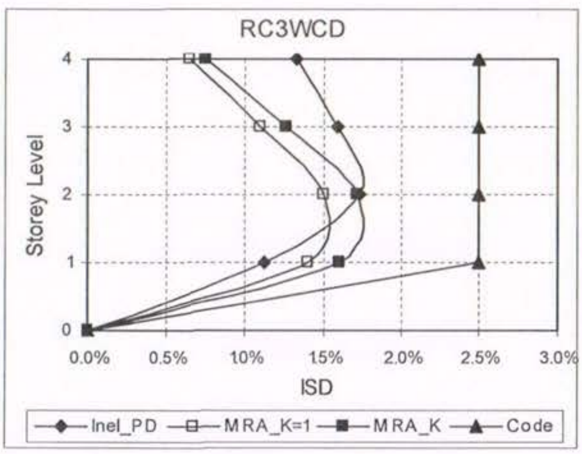


Figure 8: ISD profile for RC3WCD

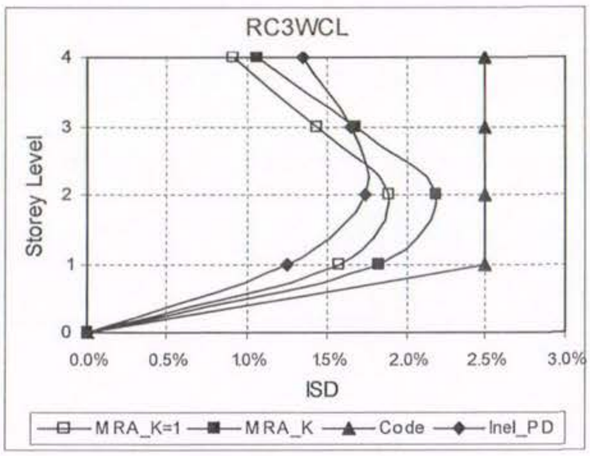


Figure 9: ISD profile for RC3WCL

The deflection profiles for these buildings (Figures 10 and 11) indicate that the inelastic deflection is close to the deflection profile from MRA with k_{dm}=1.0. This observation indicates that the deflection from MRA without the drift modification factor could be considered to represent the 'actual' inelastic displacement demand profile.

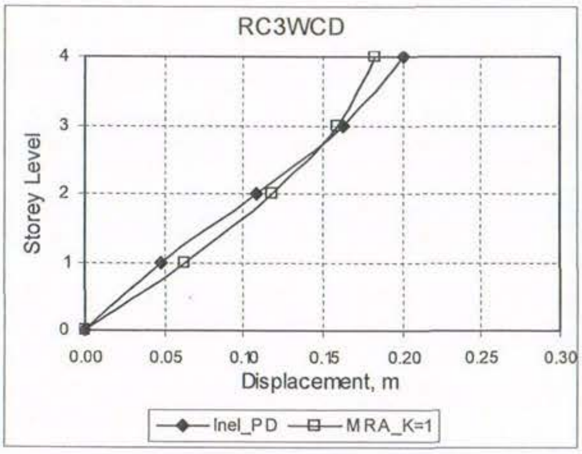


Figure 10: Deflection profile for RC3WCD

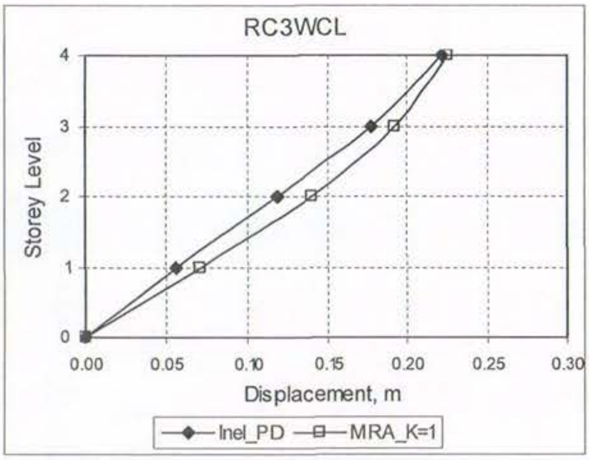


Figure 11: Deflection profile for RC3WCL

5.2.2 Wellington 3-storey Steel buildings

5.2.2.1 Ductile vs Limited Ductile Building

Figures 12 and 13 depict ISD profiles. The inelastic drift demand at the 1st storey level is greater than that in other storeys. In the case of limited ductile building, 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.

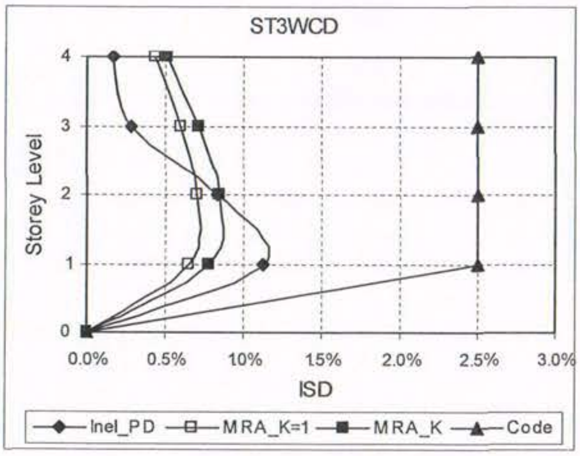


Figure 12: ISD profile for ST3WCD

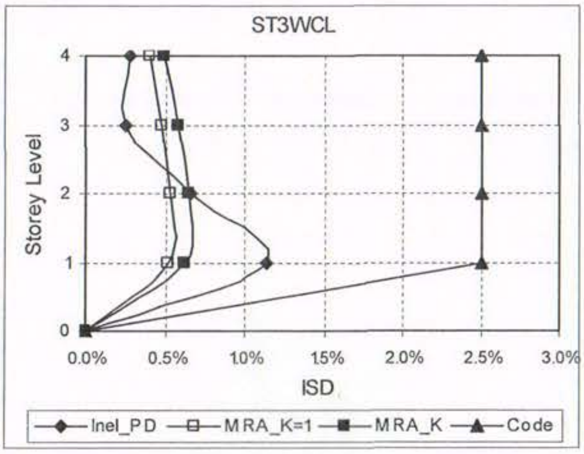
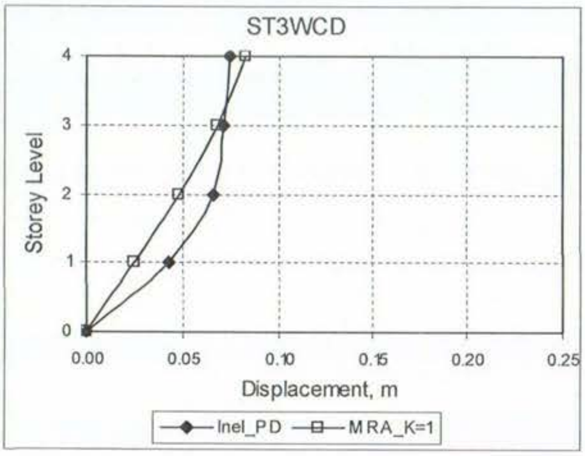


Figure 13: ISD profile for ST3WCL

Three different section sizes were used in these buildings, at (a) 1st storey level (b) 2nd and 3rd storey level (c) roof level. It should be noted that, as expected, the drift pattern changed and the ISD increased wherever the section sizes were reduced up the height of the building.

The deflection profiles in Figures 14 and 15 show (a) that the inelastic deformation exceeded MRA results at bottom storey levels, and (b) that the difference is large in the case of the ductile building.



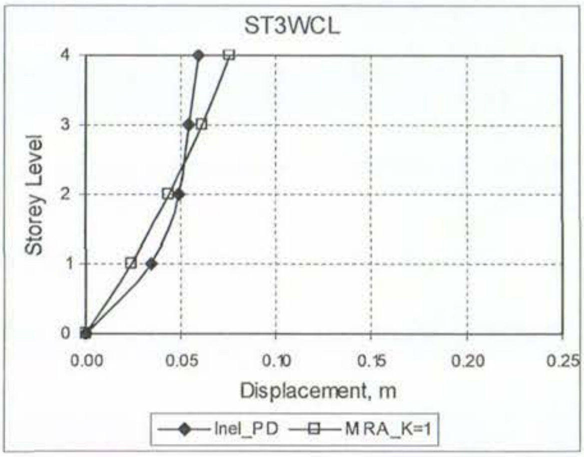
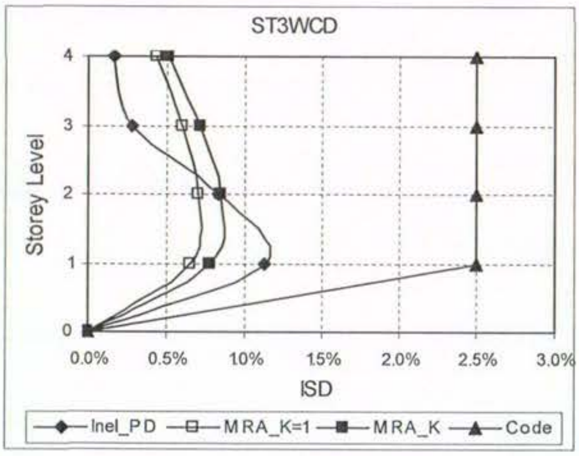


Figure 14: Deflection profile for ST3WCD

Figure 15: Deflection profile for ST3WCL

5.2.2.2 Shallow soil vs Deep soil effects

The ISD profile for deep soil was very similar to that for shallow soil (Figures 16 and 17) with the exception that the maximum ISD realised for ST3WDD was about 1.5%, compared to 1.1% for ST3WCD.



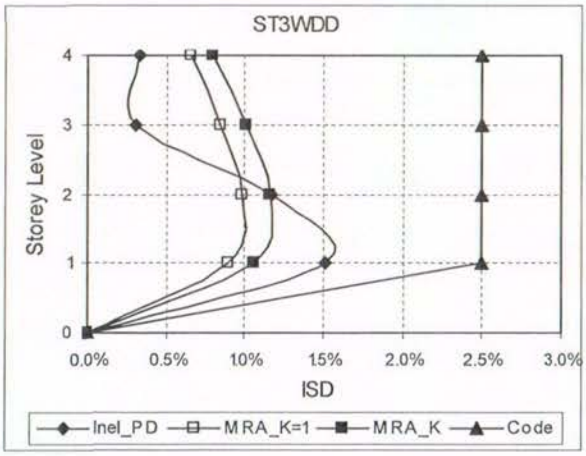


Figure 16: ISD profile for ST3WCD

Figure 17: ISD profile for ST3WDD

The deflection profiles (Figures 18 and 19) confirmed that the larger deformations were confined to the lower storeys and were larger for the D soil building. The reason can be attributed to the higher loading demand from NZS1170.5 for the deep soil class.

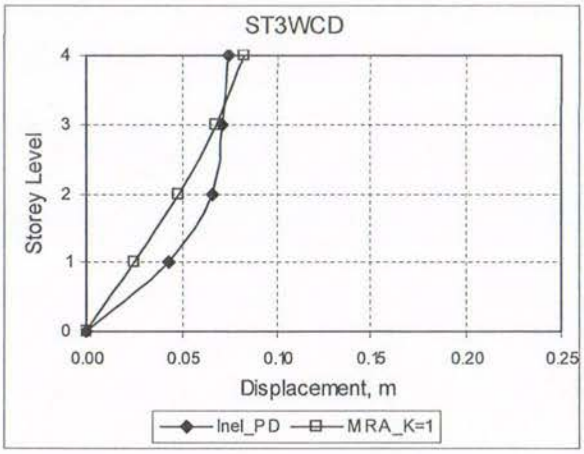


Figure 18: Deflection profile for ST3WCD

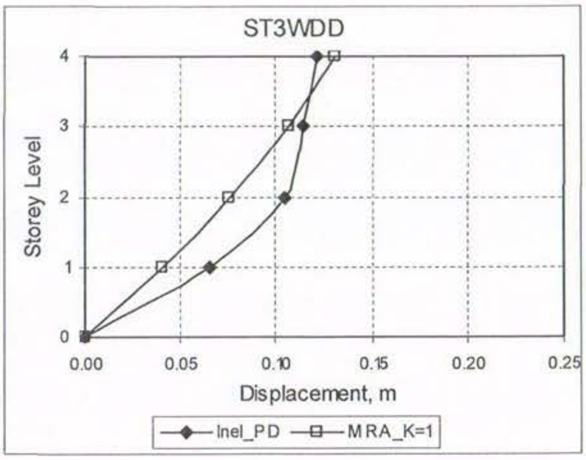


Figure 19: Deflection profile for ST3WDD

5.2.3 Auckland Buildings

Two buildings were considered for the Auckland region, both with limited ductility of 3. For RC buildings, it can be observed (Figures 20 and 22) that the ISD and deflection profiles were less than the MRA derived values. For steel buildings, inelastic ISD values exceeded MRA values only for the 1st storey (Figure 21), whereas the inelastic deflection profile (Figure 23) exceeded MRA results almost along the height of the building.

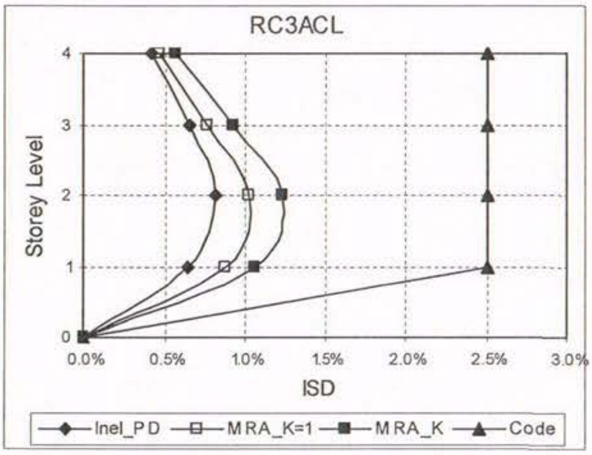


Figure 20: ISD profile for RC3ACL

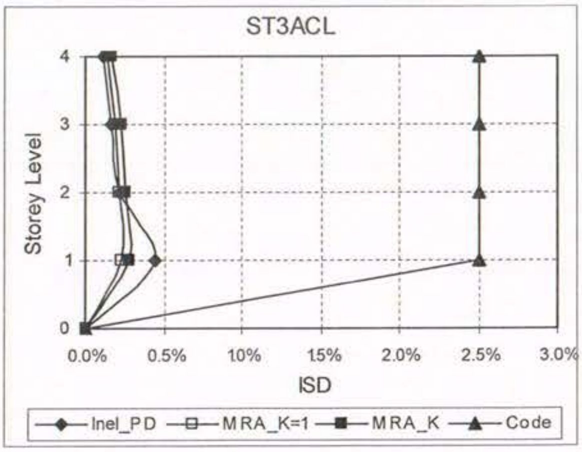


Figure 21: Deflection profile for ST3ACL

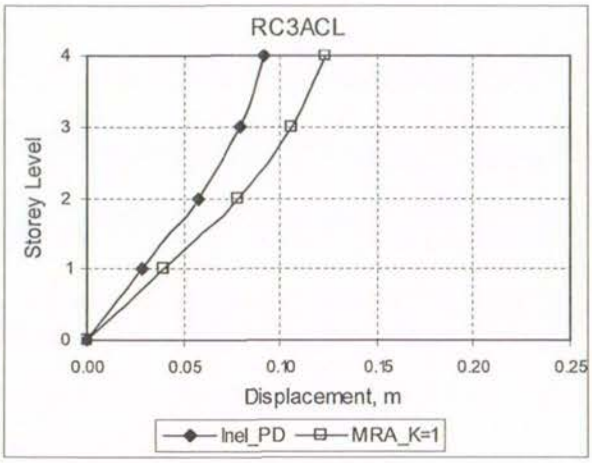


Figure 22: Deflection profile for RC3ACL

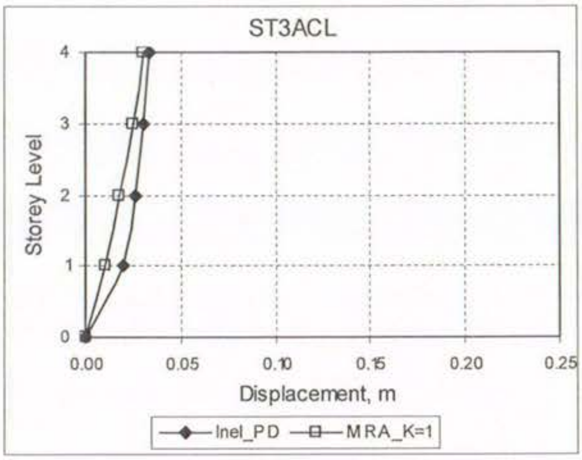


Figure 23: Deflection profile for ST3ACL

5.3 Ten-storey buildings

The responses of 10-storey buildings were compared for a group of buildings consisting of one RC frame building (RC10WCD), two RC wall buildings (RC10WCD) and (RC10WDD) and 3 steel buildings (ST10WCD), (ST10WDD), (ST10ACL). Results for frames and walls will be discussed separately.

5.3.1 RC frame buildings

In RC frame buildings, inelastic ISD values slightly exceeded MRA values at the lowest and upper floors only (Figure 24). In this building, 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. However, the maximum inelastic ISD that could be realised is only about 1.5%. The justification for inter-storey drift multiplier by $k_{dm} \neq 1.0$ remains doubtful. The inelastic deflection profiles (Figure 25) indicate that the inelastic deflection is almost equal to that from MRA at lower storeys, but significantly smaller at upper storeys.

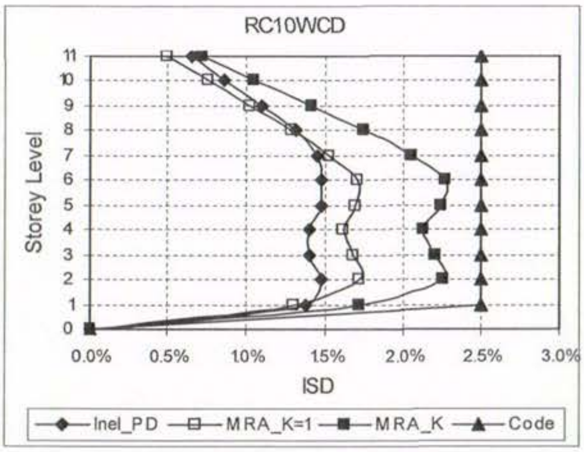


Figure 24: ISD profile for RC10WCD

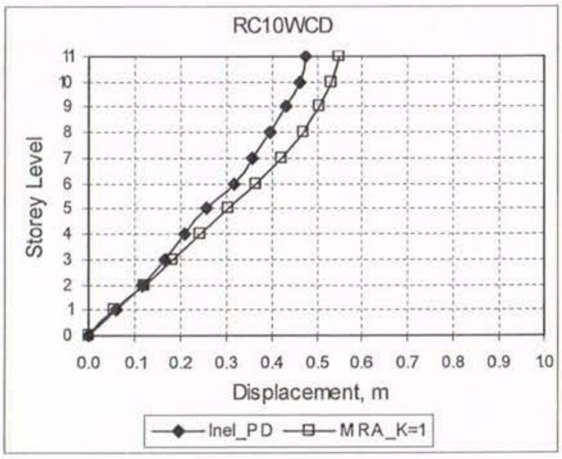


Figure 25: Deflection profile for RC10WCD

5.3.2 Steel EBF buildings

5.3.2.1 Ductile vs limited ductile building

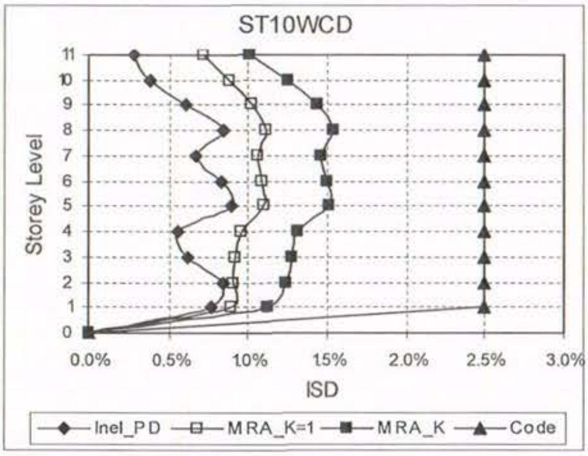


Figure 26: ISD profile for ST10WCD

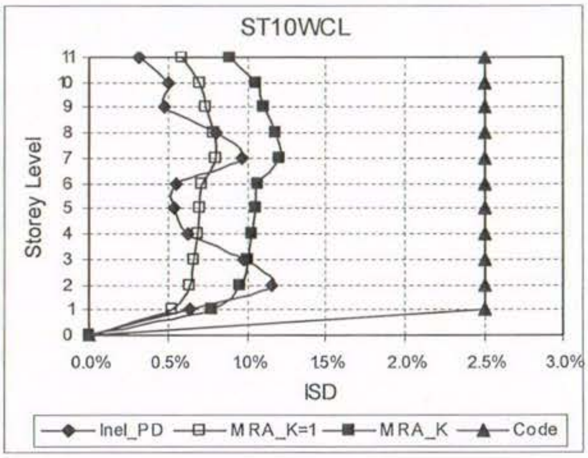


Figure 27: Deflection profile for ST10WCL

The inelastic ISD profiles for ductile and limited ductile buildings are shown in Figures 26 and 27. 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, stiffest section is at 1st storey level; member sizes were changed at 2nd storey, 5th storey and 8th storey. For ST10WCL building, very stiff section was used at 1st storey level. The reduction in member size at 2nd storey level is 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 27.

Referring to the deflection profiles (Figure 28 and 29), the inelastic deflection demand is very similar for both the ductile and limited ductile buildings, except for a higher demand at lower storey levels in the limited ductile building. Note that the ARC2 record gave extreme results for both ST10WCD and ST10WCL, and the responses it generated were considered outliers and were disregarded.

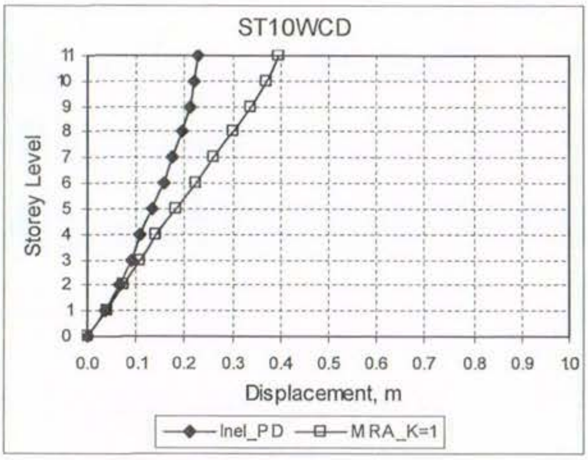


Figure 28: Deflection profile for ST10WCD

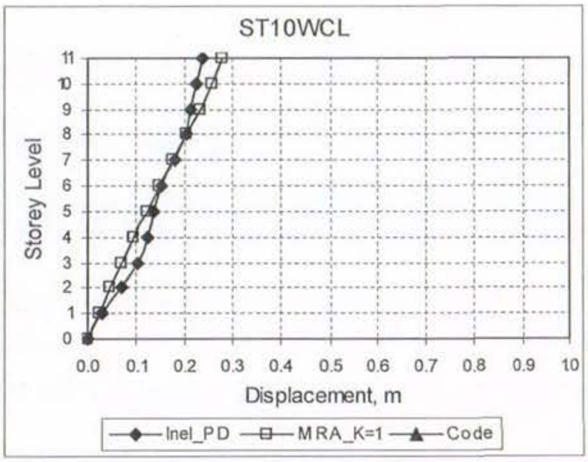


Figure 29: Deflection profile for ST10WCL

5.3.2.2 Steel building in deep soil

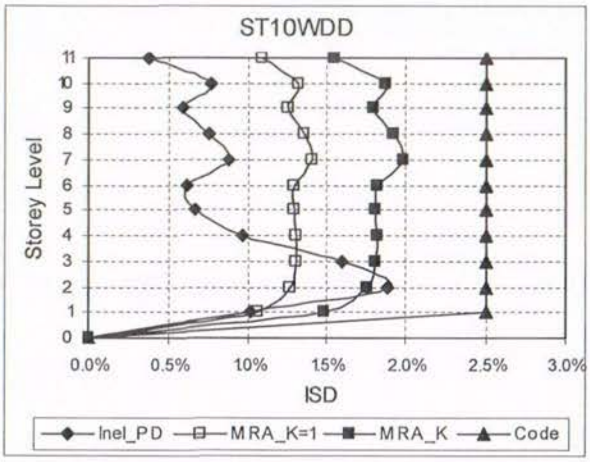


Figure 30: ISD profile for ST10WDD

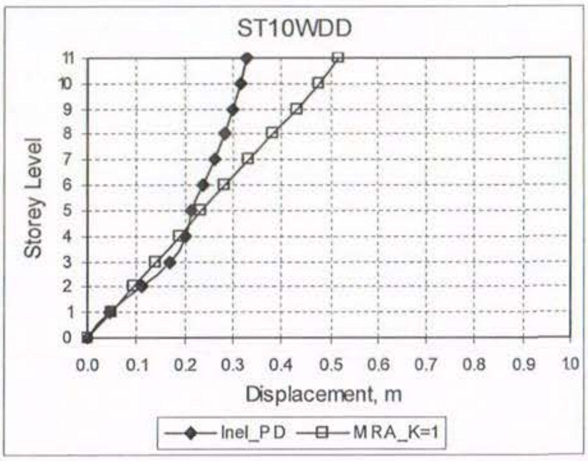


Figure 31: Deflection profile for ST10WDD

In ST10WDD building, stiffest member was used in 1st storey level and member sizes were changed in 2nd, 7th and 10th storey levels. The ISD profiles and deflection profiles are shown in Figures 30 and 31 respectively. The higher demand at 2nd lowest storey is due to drastic reduction in section size compared to the immediate lower storey level. YPT1 record gave extreme values so their results were ignored. The issue of extreme demands particularly for only a few records is discussed later in the report.

5.3.2.3 Auckland buildings

The ISD profile (Figure 32) 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 deflection profile is somewhat lower than MRA profile at all levels (Figure 33).

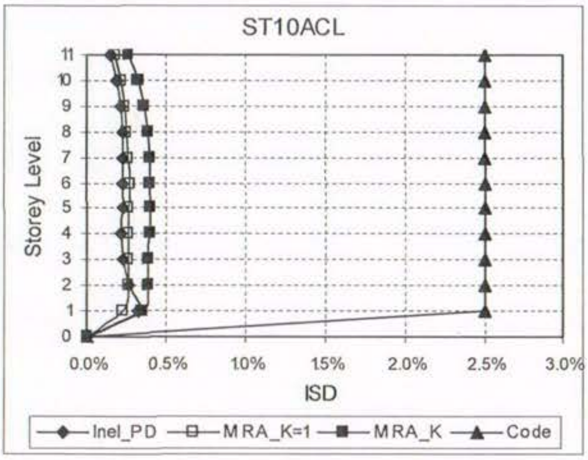


Figure 32: ISD profile for ST10ACL

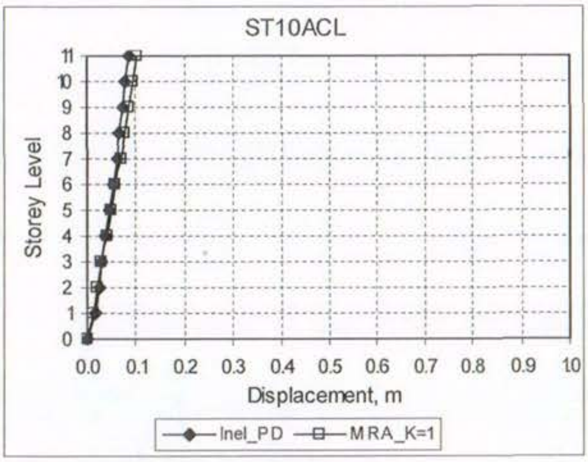


Figure 33: Deflection profile for ST10 ACL

5.3.3 Wall buildings:

Two shear wall buildings were considered, one on shallow soil (RW10WCD) and other on deep soil (RW10WDD) conditions for Wellington. Both were designed for ductility 5.

In building RW10WCD, a shear wall of size 9 m x 0.5 m was adopted. For building RW10WDD building, 2 walls of size 9 m x 0.45 m were used, which resulted in stiffer building with a period of 1.8s, compared with RW10WCD for which the period was 2.3s. Hence, the profiles of MRA results for the deep soil building are slightly less than for the shallow soil one. The structural modelling of the wall included an NL link element at the mid-point of the plastic hinge zones at the base of the wall, and rest of the wall segments were considered elastic with reduced effective moment of inertia and did not include any nonlinear elements. The moment-rotation capacities of the NL link elements were derived based on the expected capacity of the section. Takeda rules were adopted to simulate the cyclic response.

The inelastic ISD demands are shown in Figure 34 and Figure 35. The ISD profile is markedly different from the MRA results. For it, the maximum contribution is provided by the bottom 1/3rd of the total wall height, and for the rest of the wall height the ISD demand remains almost constant. This pattern is reasonable because of the concentrated rotation of the wall at the base. This trend is further exhibited in the deflection profiles of the wall (Figures 36 and 37), which are basically very similar to the deflected shapes of cantilever walls. The inelastic ISD profiles for both buildings are again very similar. However, it should be noted that for building RW10WDD, the record YPT1 resulted in very high demands. Ignoring its contribution, the average ISD was determined only from other 6 records.

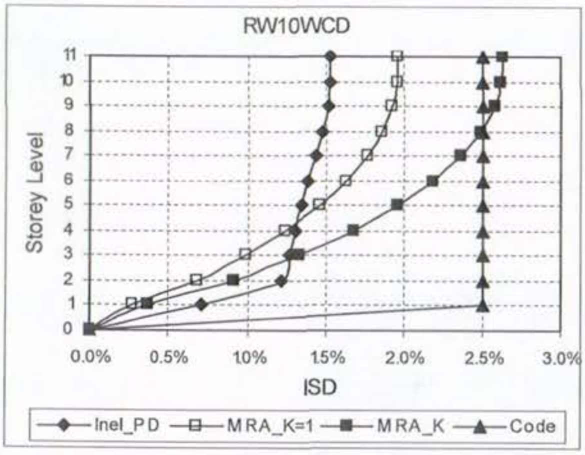


Figure 34: ISD profile for RW10WCD

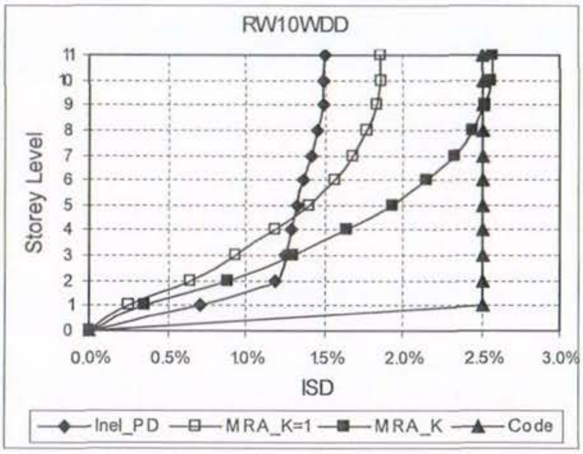


Figure 35: ISD profile for RW10WDD

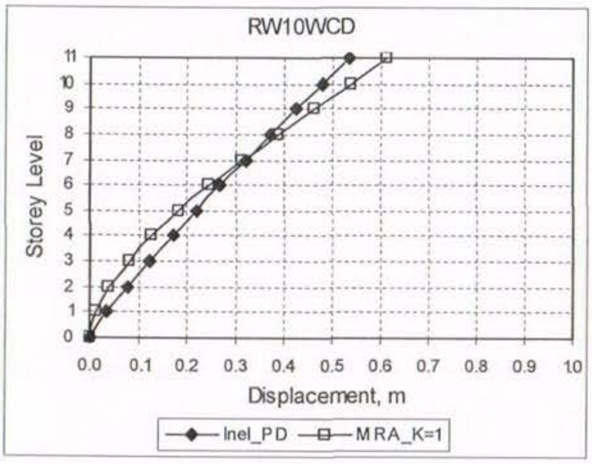


Figure 36: Deflection profile of RW10WCD

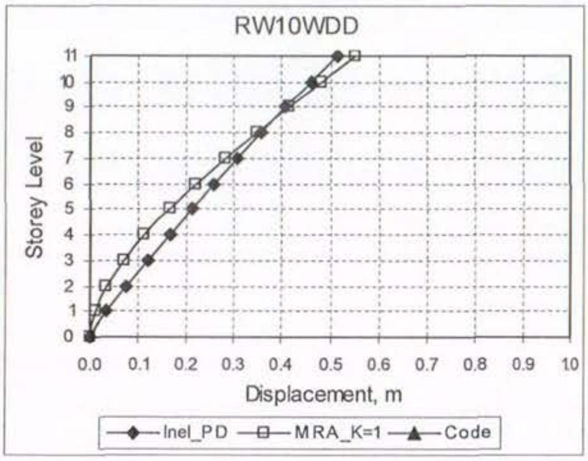


Figure 37: Deflection profile of RW10WDD

5.3 Twenty-storey wall building

For the 20-storey wall building, a wall section of 18 m x 0.5 m was considered. The natural period of the building was about 3 seconds which is in the long period range. As per the design spectrum, the force demand is less in this region compared to short period ranges. The inelastic drift demand for this building can be seen to be close to the inelastic drift demands realised by 10-storey wall buildings. This can be attributed in part to the reduced input demand from the records at the long period and in part to the rotational capacity associated with the wall section.

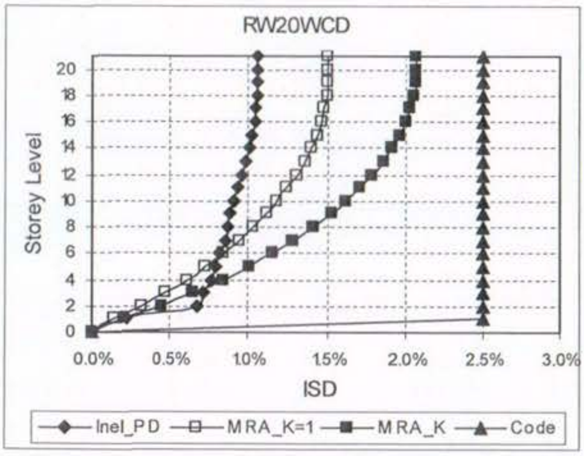


Figure 38: ISD profile for RW20WCD

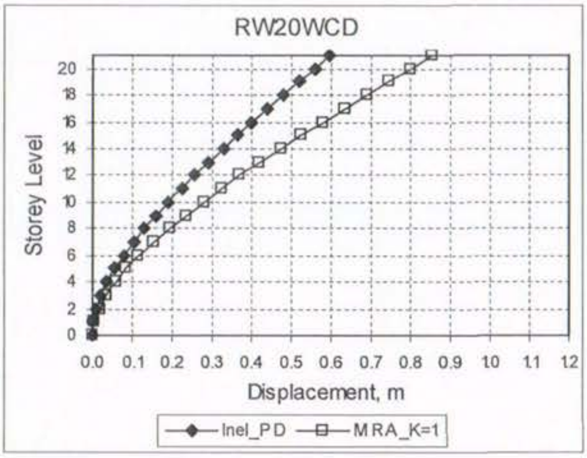


Figure 39: Deflection profile for RW20WCD

For the 20-storey wall building, a maximum of 0.75% ISD demand was realised at the bottom storey level, and the maximum ISD higher up the building was about 1.0%. At the 10^{th}-storey level of the 20-storey building, the inelastic displacement was only 0.2 m, which is less than the ½ of top storey value. Note that the maximum inelastic displacement at the top level is close to that of the 10-storey wall building. With the reduced average ISD, the taller 20-storey wall building reached a similar level of displacement at the roof.

5.4 Scale factors from the present study

The purposes of the present study were (i) to confirm the published scaling procedure for drifts obtained from MRA procedures and (ii) to suggest rational scaling procedure, if necessary. The following discussions are done separately for Wellington and Auckland buildings with respect to their numbers of storeys and structural systems.

The scale factor is derived as the ratio of ISD from inelastic time-history analyses to that from MRA procedures without any drift modification factor, as denoted by (MRA_K=1) in many the figures above (for example, Figure 38).

In 3-storey low-rise RC frame buildings, the ratio increased up the height of the building (Figure 40). The maximum average value is 1.25.

For steel buildings, Figure 41, the scale factor profile was very different from that for RC buildings. The ratio sometimes exceeded 2.0 as in the case of ST3WCL. The individual values of ISD for the inelastic case varied within a range of 1.15% to 1.5%, and MRA values varied between 0.5% and 0.8%. The ratios for all three buildings were less than 1.0 in the upper floors, indicating inelastic ISD demands not exceeding MRA values. From these results, it may be suggested that for MRA, k_{dm} of 1.5 is appropriate over the bottom $1/3^{rd}$ of the total height of the building only, with k_{dm}=1.0 being used elsewhere.

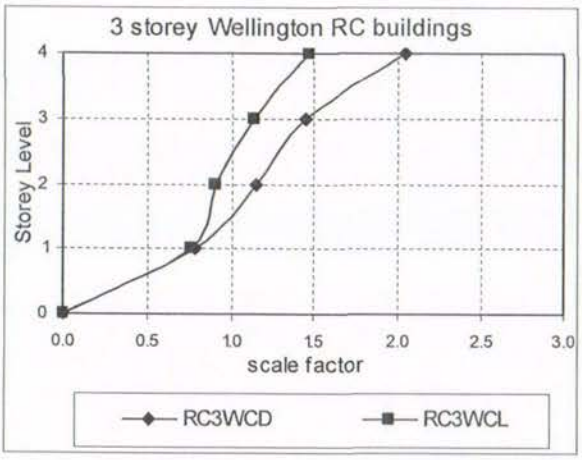


Figure 40: Ratios for RC 3-storey Wellington

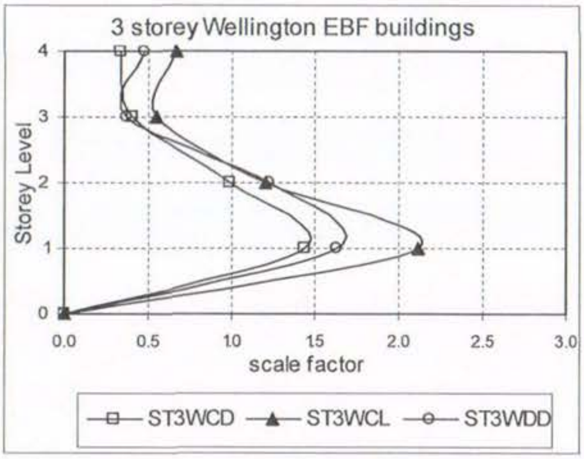


Figure 41: Ratios for Steel 3-storey Wellington

Figure 42 shows the scale factors ratios derived for Auckland buildings (RC and steel). For the RC building, it is clear that k_{dm}=1.0 for this form of structures in lower storey zones. On the other hand, for the steel building, a higher ratio is depicted at the bottom storey level. This high value is due to the ratio of two small ISD values at the lowest storey level. So, for these buildings, kdm of 1.5 may be appropriate at lower levels.

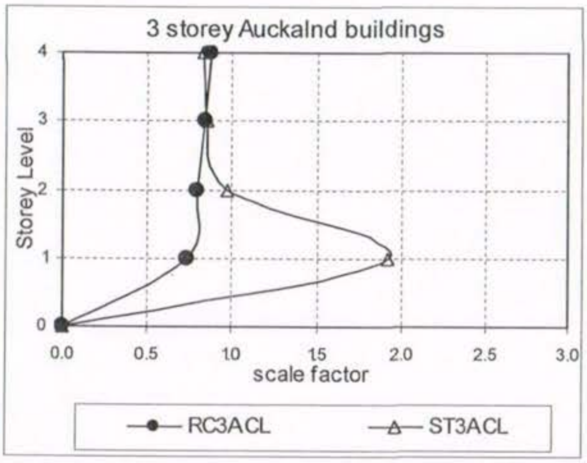


Figure 42: Scale factors for Auckland buildings

5.4.1 10-storey Frame buildings in Wellington

Figure 43 shows the ratios up the height of an RC 10-storey building on class C soil. The ratio slightly exceeded 1.0 at lowest storey and again in the upper storeys. The recommended scale factor is, therefore, 1.25 as in the case of 3-storey RC frame building. Similarly from Figure 44, a scale factor of 1.5 is recommended at the lowest 1/3 storey height and 1.0 may be recommended for the rest of the height.

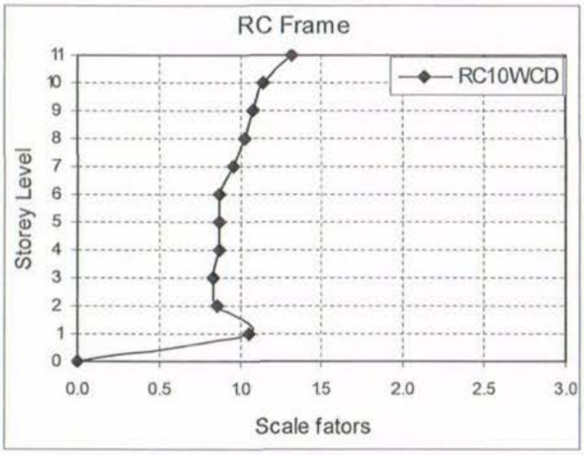


Figure 43: Ratios of RC 10-storey, Wellington

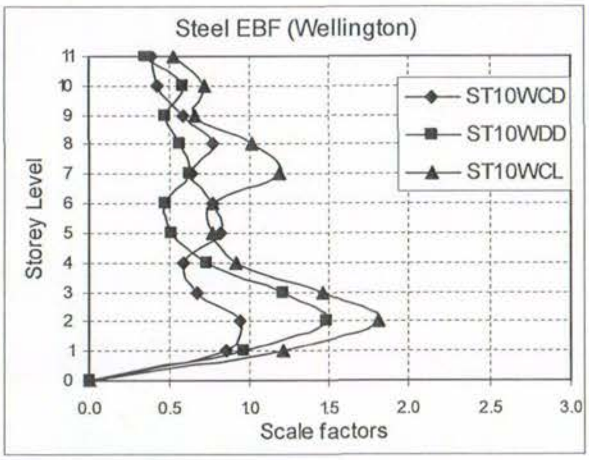


Figure 44: Ratios of Steel 10-storey, Wellington

5.4.2 10-storey Auckland building

The ISD ratios (Figure 45) suggest that using a scale factor equal to 1.0 in the upper floors and 1.5 in the lower two floors are appropriate.

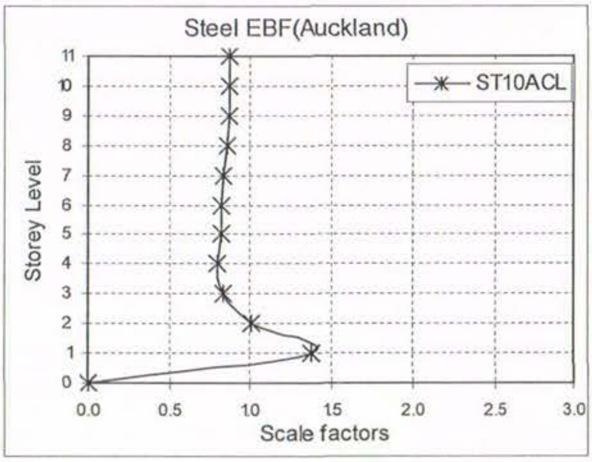


Figure 45: Ratios for Steel 10-storey, Auckland

5.4.3 Wall buildings in Wellington

The scale factors obtained for 10-storey and 20-storey wall buildings are given in Figures 46 and 47. It may be seen that the higher values are at the bottom storey/s (to where the plastic hinge zone extended). Again, this large scale factor is due to the ratio of two small numbers at that location, where the actual ISD values were considerably less than the code limit. Hence, for practical reasons of design, a scale for of 1.5 is suggested at the bottom of the wall for a height equal to twice the plastic hinge zone and 1.0 for rest of the height of the building.

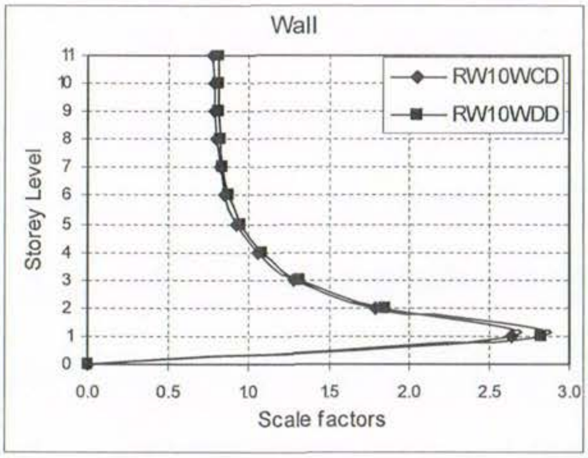


Figure 46: Ratios for Wall 10-St., Wellington

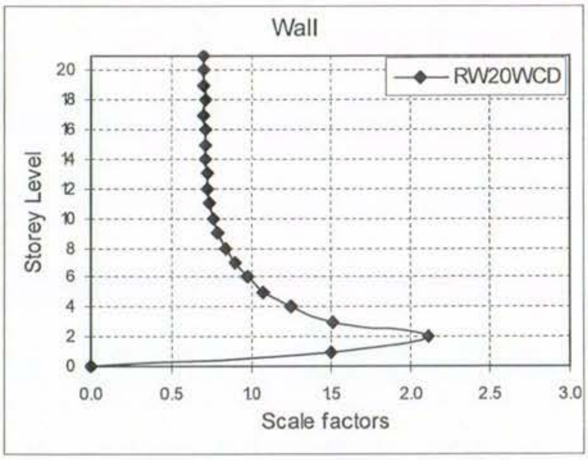


Figure 47: Ratios for Wall 20-St., Wellington

5.5 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. The buildings and the corresponding records showing such anomalies are listed in Table 5.

Table 5: List of buildings and record combinations that resulted in extreme responses

SI No	Building	Records eliminated
1	RC3WCD	K0392
2	ST3WCD	LUC1
3	RW10WDD	YPT1
4	ST10WCD	ARC2
5	ST10WCL	ARC2
6	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 rest of the records considered in the suite. To explore the reason, it was decided to examine the matching of spectral responses of the records with the NZS target spectrum. The following paragraphs discuss this issue with example buildings.

5.5.1 Frame building

The elastic fundamental period of building RC3WCD was 1.1s. Out of 7 records considered and matched with the target spectrum, records LUC1 and ARC2 gave highest responses and for record K0392, convergence failure was encountered. The ISD and displacement profiles include LUC1 and ARC2 but not K0392. Figure 48 shows response spectra for all of the records considered, and the NZS target spectrum, for the case of shallow soil (C) in Wellington. The matching procedure considered a range of periods from $0.4T_1$ to $1.3\ T_1$ within which the fitting was done. It can be appreciated that the records LUC1, ARC2 and K0392 exceeded the target spectrum at periods beyond the scaling range, as shown in Figure 49. The building period is expected to extend due to its inelastic deformation and hence the records which exceed the target spectrum in the extended period range tend to result either in higher responses or in convergence failure during ITHA. Also, the previous cyclic history of the building for these records could also be a possible influencing parameter for extreme responses for the records which exceed the target spectrum. The ISD profile for RC3WCD without the above three records is shown in Figure 50 which essentially shows elastic behaviour.

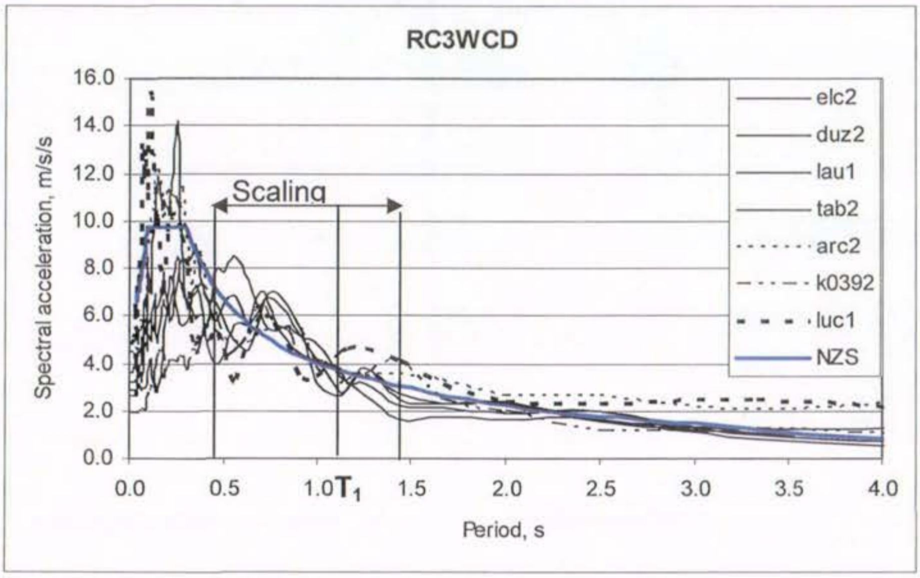


Figure 48: Spectra matching of chosen records with NZS target spectrum

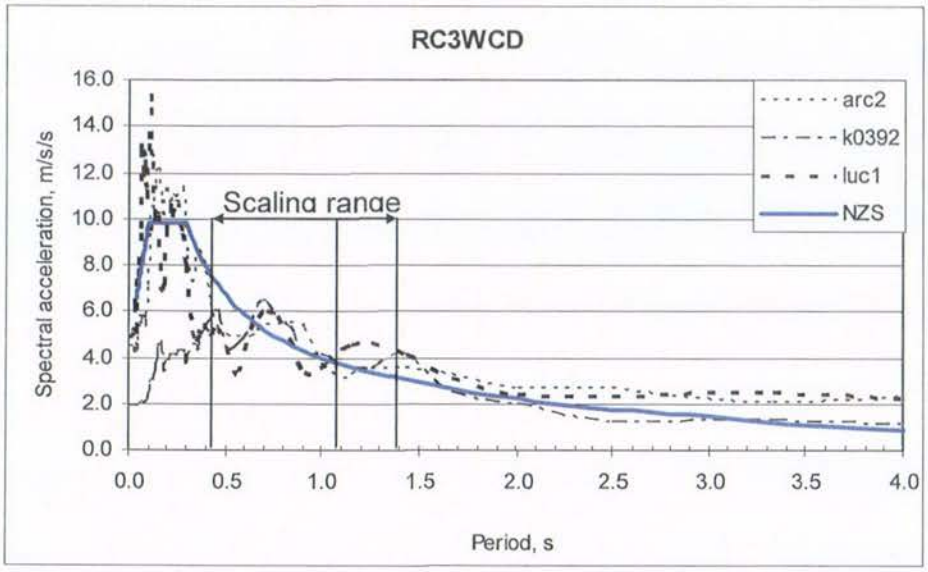


Figure 49: Response spectra matching for LUC1, ARC2 and K0392

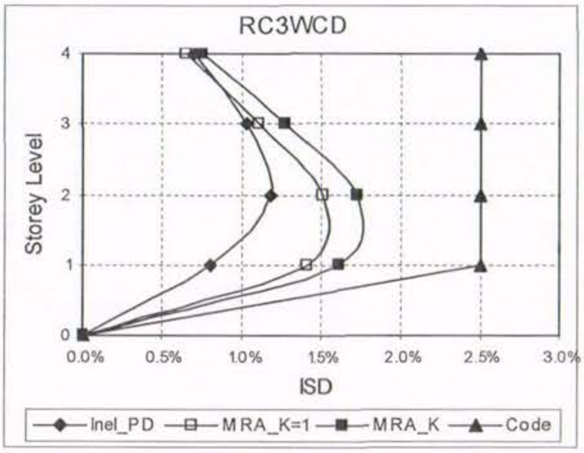


Figure 50: ISD profile without extreme records

5.5.2 Wall building

In wall building RW10WDD, highest responses were observed with the YPT record. The elastic fundamental period of building RW10WDD was 1.8s. The matching of spectra is shown in Figure 51. Out of all the records considered, YPT1 resulted in extreme responses. The possible reason can be along similar lines as explained for building RC3WCD. However, the other component YPT2 also exceeded the target spectrum as shown in Figure 52.

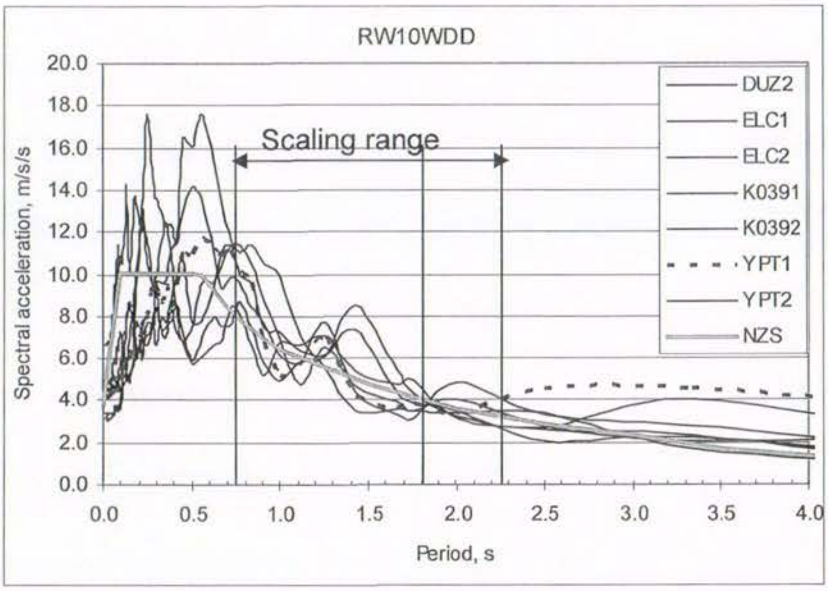


Figure 51: Response spectra matching for all chosen records

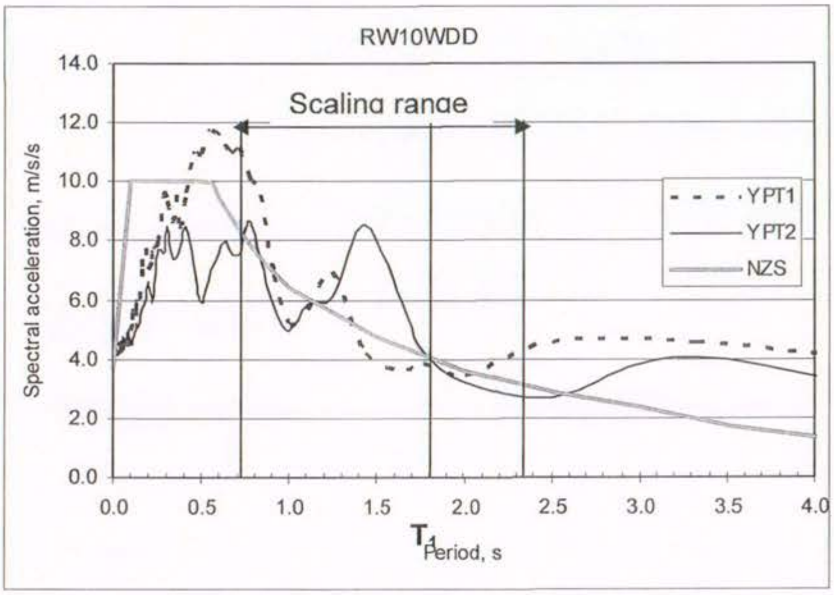


Figure 52: Response spectra matching for YPT records

The numerical results for record YPT1 was unacceptably large and was not considered. The other component YPT2 also resulted abnormally high demands compared to rest of the records, for example DUZ2, in the group. The moment-rotation demands at the base of the wall under YPT2 and DUZ2 are shown in Figure 53 and Figure 54 respectively. The smaller demand for DUZ2 and higher demand for YPT2 are evident.

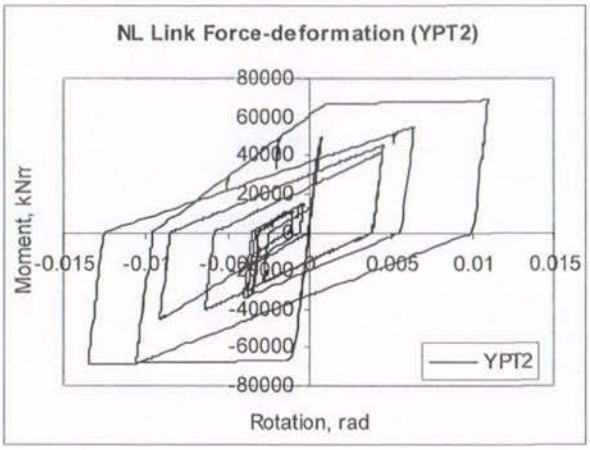


Figure 53: NL Link demands (YPT2)

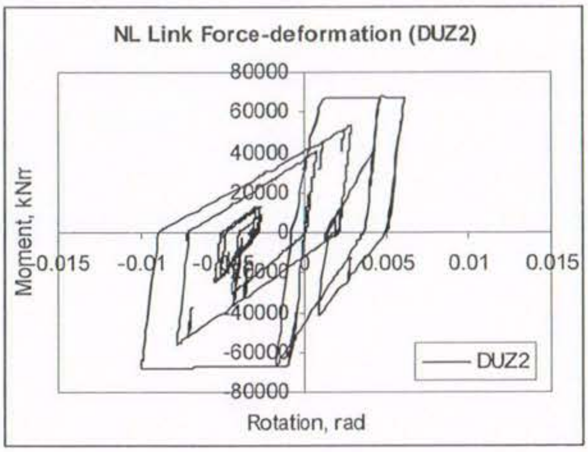


Figure 54: NL-Link demands (DUZ2)

NL Link force-deformation has been plotted for YPT2 which exceeded the target spectrum at longer periods and DUZ2 which was less than the target spectrum. These variations consequently affected the degree of inter-storey drifts at the bottom storey levels.

Figure 55 shows three ISD profiles obtained for (i) records which were less than target spectrum (T.S), (ii) records exceeding T.S., in this case YPT2, and (iii) the average of all the records. This observation cautions that the designer would face difficulty in deciding the records for analysis and design purposes.

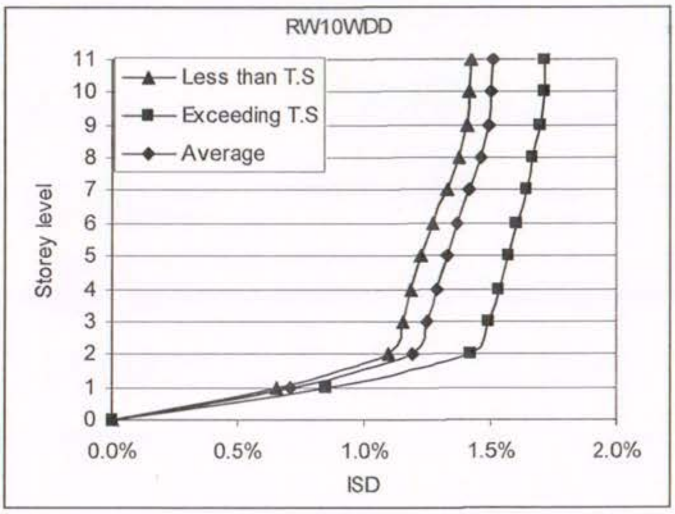


Figure 55: Influence of extreme responses in ISD profiles

6.0 SUMMARY AND CONCLUSIONS

The main objective of this research was to compare the inter-storey drifts arrived from Inelastic Time-History Analysis (ITHA) with those from Modal Response Analysis (MRA) and to verify the extent of amplification of responses. The current provisions in NZS1170.5 recommend using a drift modification factor, k_{dm} to amplify inter-storey drifts calculated by

MRA procedures at every storey level. The extent of amplification depends on the height of the building. In this study, a suite of buildings were chosen and were subjected to different sets of earthquake records appropriate for the location and soil conditions. Observations from this study are presented below:

- The design of buildings were carried out to achieve a target inter-storey drift ratio of 2.5% which is the maximum limit suggested in the standard. However, certain requirements from material standards precluded the achievement of this limit. Hence, for many buildings the linterstorey drift values fell well below the code limit.
- P-Delta forces for design were calculated using only Method B for all the buildings considered. Note that the possibility of this method being a source of difference between the interstorey drifts derived from Response spectrum method and those from ITHA, has not been investigated in this study.
- In the case of 3-storey buildings, inelastic inter-storey drift profiles for RC buildings were markedly different from those of steel buildings. The ISDs realised in steel buildings were smaller than those realised in RC buildings. Larger ISDs were concentrated in the lowest storey (1^{st} storey level) for steel buildings but in the 2^{nd} and 3^{rd} storeys for reinforced-concrete buildings.
- The ISD profiles for RC buildings from ITHA analyses basically followed the ISD profiles derived from MRA, however the drifts exceeded MRA values at upper storeys.
- ISD values obtained from ITHA for RC ductile building RC3WCD and limited ductile building RC3WCL did not show significant difference.
- The ISD at the lowest storey of steel ductile building (ST3WDD) located in deep soil was higher than that exhibited by ST3WCD located in shallow soil.
- The ISD profiles in 10-storey steel buildings greatly exceeded the MRA values at lower storeys but not in upper storeys.
- ISD profiles for 10-storey ductile and limited ductile steel buildings were almost the same for both patterns and values.
- Inelastic inter-storey drift profiles for wall buildings were markedly different from those derived by MRA procedures.
- Maximum drifts were concentrated within the lowest $1/3^{rd}$ of the wall height, and the drifts almost remained constant over the upper levels.
- Inelastic displacement profiles for 3-storey RC ductile and limited ductile buildings were close to elastic profiles.
- Inelastic displacement profiles for 3 storey steel buildings in Wellington exceeded the elastic profiles, at lower storeys only.
- For Auckland sites, and for RC buildings the inelastic responses were well below the MRA results over the heights of the buildings. However for steel buildings, inelastic drifts were double MRA values. Hence, the inelastic displacement profiles for steel buildings in Auckland exceeded the MRA profiles.
- Even though the ISD values from ITHA were more than those derived from MRA, all values were within the prescribed code limit of 2.5%

Recommendations from this study:

The following revised drift modification factors are suggested for different structural systems located in high and low seismicity areas (Table 6).

Table 6: Recommendations for revised drift modification factors.

Type of structure	Location	Recommended Drift modification factor
Low-rise RC Frame building	Wellington	1.25
Low-rise RC Frame building	Auckland	1.0
Low-rise steel building	Wellington	1.5
Low-rise steel building	Auckland	1.5 in lower two storeys and 1.0 elsewhere
High rise RC Frame building	Wellington	1.25
High rise steel building	Wellington	1.5 in lower 1/3rd height and 1.0 elsewhere
High rise steel building	Auckland	1.0
High rise shear wall building	Wellington	1.5 for a length equal to twice the plastic hinge zone of the building and 1.0 elsewhere

7.0 ACKNOWLEDGEMENTS

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