SEISMIC PERFORMANCE of the DIAPHRAGMS of BUILDINGS – PHASE 1: DETERMINING THE LATERAL EARTHQUAKE FORCES FOR DESIGN OF DIAPHRAGMS

For MBIE/UOC

REPORT under UOC-6378

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Holmes Consulting

Holmes Consulting

Level 2, 254 Montreal Street
Christchurch Central
PO Box 6718
Christchurch 8442
holmesgroup.com

Report

UOC MBIE RESEARCH PROGRAMME – SEISMIC PERFORMANCE of the DIAPHRAGMS REINFORCED CONCRETE BUILDINGS

Prepared For: Ministry of Business, Innovation and Employment, and the University of Canterbury

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Prepared By:

Des Bull
TECHNICAL DIRECTOR
Holmes Consulting LP

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Holmes

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1 EXECUTIVE SUMMARY

The findings of two independent studies by PTL Structural Consultants (PTL) and Holmes Consulting LP (HCLP) have **validated** the use of an earthquake induced lateral force distribution, develop by Dr D. Gardiner (at University of Canterbury). The method for determining the lateral force distribution for designing the diaphragms of buildings is known as "the pseudo-Equivalent Static Analysis (pESA)". This method has been incorporated into:

- The NZS 1170.5:2004, Cl. 5.7 and Commentary (C5.7 and Appendix A to C5.7).

- "*The Seismic Assessment of Existing Buildings* (*the Guidelines*)", July 2017, managed jointly by the Ministry of Business, Innovation and Employment, the Earthquake Commission, the New Zealand Society for Earthquake Engineering, the Structural Engineering Society and the New Zealand Geotechnical Society: Sections C2E.3 and C2E.5.

NZS1170.5 and "the Guidelines" limit the height of a building for use of pESA to 9 storeys and with diaphragms formed of reinforced concrete. The two studies appended here have confirmed that "for general" use that the pESA is suitable for structures 9 storeys or less in height. When the shape of the structure is dominated by the first translatory mode (for the direction of earthquake attack that is being investigated) the pESA may be applied above 9 storeys. The research also concluded that the method can be used for timber structural floors, providing the stiffness of the timber diaphragm is accounted for in the analysis.

The other methods of determining the forces within diaphragms were investigated. The methods either under-predicted (ASCE 7-10, or Sabelli's method) or excessively over-predict (Parts and Portions, NZS 1170.5) the floors accelerations as compared to the pESA method. Some methods fail to recognise that design of diaphragms involves the 3-dimensional structure with interactions between the vertical lateral force resisting structures (moment frames, walls or braces) both horizontally and vertically. This lack of 3-D investigation will underestimate the forces across diaphragms (Cowie et al, 2014).

2 INTRODUCTION

Holmes Consulting LP have been engaged by the University of Canterbury (main contractor) to the Ministry of Business, Innovation and Employment to undertake research in to one aspect of the seismic performance of the concrete diaphragms (floors) of reinforced concrete buildings.

This research project is to review some of the various methods employed internationally and in NZ for determining the lateral force distribution to floor diaphragms, resulting from earthquakes that would allow a reasonable estimate of the forces generated in diaphragms. Particular focus of the research was a comparison of the various methods to the method now in NZS 1170.5 and Commentary, the "pseudo-Equivalent Static Analysis" (pESA).

To facilitate this research, PTL was contracted by the UC Quake Centre, to work with HCLP to develop a scope of analytical study and report those findings. The PTL research study compared peak floor accelerations for a range buildings (7 of) with the lateral force distributions from the references below, to that reported from non-linear time history analysis (NLTHA) of those structures. The PTL report is included in the Appendices.

The study undertaken by PTL, used the following referenced methods for determining lateral forces to be applied to floor diaphragms for the design of the diaphragms:

- NZS 1170.5:2004 – Structural design actions – Part 5: Earthquake actions – New Zealand

- NZS 1170.5 Supp1:2004 – Structural design actions – Part 5: Earthquake actions – New Zealand Commentary
    - Including for the sake comparison , the forces determined by Parts and Components of NZS 1170.5
- ASCE 7-10 – Minimum design loads for buildings and other structures
- EN-1998-1:2004 – Eurocode 8 – Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings.

HCLP embarked on a similar research study, comparing the peak floor accelerations from NLTHA of 12 buildings to the requirements for floor forces from NZS1170.5:2004. The HCLP NLTHA report in included in Appendices.

3 SCOPE OF WORK

The scope of work for this project included the following:

1. Review a number of recommended methodologies from around the world, for determining the floor accelerations (forces) applied to floor diaphragms of buildings, during significant earthquakes:

    - ASCE 7-10 and ASCE 7-16 – Minimum design loads for buildings and other structures
    - Cowie, K.A., Fussell, A.J., Clifton, G.C., MacRae, G.A., and Hicks, S.J., 2014, “Seismic Design of composite metal deck and concrete-filled diaphragms – A discussion paper”, NZSEE Conference 2014.
    - Sabelli, R., Sabol, T., and Easterling, W., 2011, “Seismic Design of Composite Streel Deck and Concrete-filled Diaphragms”, National Institute of Standards and Technology, NIST GCR 11-917-10.
    - NZS 1170.5:2004 – Structural design actions – Part 5: Earthquake actions – New Zealand
    - NZS 1170.5 Supp1:2004 – Structural design actions – Part 5: Earthquake actions – New Zealand Commentary
    - EN-1998-1:2004 – Eurocode 8 – Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings.

2. Computer modelling would be done for a range of building configurations, types of construction (frames, walls and a combination of these) and different construction materials (timber, concrete and steel). Comparisons would be made of the output peak floor accelerations with the recommendations for peak floor accelerations of the various methods being reviewed. By doing so, the applicability of the methods can be judged, in terms of closeness of fit with the analysis outputs.

    The computer analysis method chosen because it can represent the plastic behaviour of the building components, was a non-linear time history analysis (NLTHA). The NLTHA will help determine:

    i. Can the pseudo-Equivalent Static Analysis method (pESA) can be applied above 9 storeys (a limitation suggested in NZS 1170.5:2006)?

    ii. What is the effect on the magnitude and interstorey distribution of floor forces by applying Peak Ground Acceleration (PGA) up the full height of a structure – a result of using the pESA with flexible structures?

iii. How applicable is the pESA method to timber/flexible diaphragms?

3. Report on our findings and recommendations.

4 LIMITATIONS

Findings presented as a part of this research project are for the sole use of the client. The findings are not intended for use by other parties, and may not contain sufficient information for the purposes of other parties or other uses.

Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practicing in this field at this time. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

5 REVIEW A NUMBER RECOMMENDED METHODOLOGIES FROM AROUND THE WORLD, FOR DETERMINING THE FLOOR ACCELERATIONS (FORCES) APPLIED TO FLOOR DIAPHRAGMS OF BUILDINGS, DURING SIGNIFICANT EARTHQUAKES

5.1 Forces that develop in floor diaphragms

The forces in floor diaphragms can be developed as the building displaces during an earthquake. One contribution to the floor forces is the "inertia" of the masses at each level. During significant shaking, the vertical lateral force resisting systems can yield and then strain harden. This effectively increases the strength of the lateral force resisting elements, resulting in higher floor forces than the traditional Code-based Equivalent Static Force (ESA) design method predicts (NZS 1170.5:2004). This effect of increased strength is known as "Overstrength".

Often the vertical lateral force resisting structures/elements would have quite different deformed shapes if these were deforming individually and not linked together by the floors. This constraining of the vertical structural elements to a common deformed shape can produce very large forces within the floor diaphragms. These are known as "transfer forces".

The force/stress distributions resulting in diaphragms and through the structure are the combinations of inertia forces and "transfer forces". The inertia forces and transfer forces are mathematically coupled. This means that inertia forces cause the building to deform laterally, it is the lateral deformations that cause transfer forces. The total distribution of forces across floor diaphragms and up and down the building are inter-related and must be in equilibrium. Therefore it is not appropriate to determine the inertia forces and transfer forces separately and combine later.

5.2 Methods for determining forces in diaphragms during earthquakes

The comments and comparisons made in the following sections relate to whether or not the prescribed methods determine forces that can be used for structurally sound and cost-effective design an actual building.

- **American Society of Civil Engineering: ASCE 7-10 and ASCE 7-16 Standards**

The proposed "general" method of ASCE 7-10 and ASCE 7-16 – underestimates the floor accelerations as no accounting for building developing overstrength is made for the inertia forces. Also these Standards are, in effect, designing one floor at a time, not consideration the vertical interaction of the lateral force resisting elements.

Inertial floor forces are based on an Equivalent Static Analysis methods.

Overstrength actions are required for the collectors and their connections in to the frames and walls. The size of the Overstrength [Factor] is prescribed, and not determined specific to a building (as per NZS 1170.5). This is a potential source of error, underestimating the lateral forces on the diaphragms. Further, the method does not allow for horizontal and vertical redistribution of forces with a structure; where NZS 1170.5 does.

In ASCE 7-16, where the building has certain irregularities in terms of load paths, then the amplified transfer forces can be added to the floor inertia forces. This is not used for general cases.

- **Cowie, K.A., Fussell, A.J., Clifton, G.C., MacRae, G.A., and Hicks, S.J., 2014, "Seismic Design of composite metal deck and concrete-filled diaphragms – A discussion paper", NZSEE Conference 2014.**

The method proposed in this paper is based the "Parts and Portions" methods of NZS 1170.5:2004. It only applies to buildings without transfer effects in the diaphragms. These being simple, regular floor plates with a symmetric distribution of lateral force resisting structures across the plan of the building. There is a modification of the maximum floor acceleration coefficient (compared to NZS 1170.5), limiting it to 1.6.

The results of the inertia from this methods appear to be equal to or larger than the pseudo-Equivalent Static Method of NZS 1170.5.

The proposed method results in larger inertia forces on each floor than ASCE 7-10.

The recommendation of separately determining the inertia force distribution and the transfer effects then combining by the method: square root of the sum of the squares (SQSS) is incorrect. This method does not correctly determine the size nor direction of forces entering or leaving the floor diaphragms, at columns or beams of a moment frame, or at walls, or at braces. The outputs of the SQSS method are not in equilibrium nor of the correct vector sense.

We would not be recommending the use of the proposed method.

- **Sabelli, R., Sabol, T., and Easterling, W., 2011, "Seismic Design of Composite Streel Deck and Concrete-filled Diaphragms", National Institute of Standards and Technology, NIST GCR 11-917-10**

The method proposed by Sabelli et al recognises the increased inertia on each floor as a result of dynamic amplification within each diaphragm. The amplification of the floor forces is similar to that employed by NZS 1170.5:2004 – "Parts and Components". The method is to undertake a lateral force analysis using the Equivalent Static Analysis method (approximating the first mode of vibration force distribution) while applying the amplified force at a specific floor, the one under investigation. This approach is done for each floor. Therefore if it is a 10 storey building then the analysis is done 10 times.

The primary limitation of the proposed method is that the Equivalent Static Analysis (ESA) underestimates the transfer forces that can form. The ESA does not use the overstrength capacity of the building that is developed during a major earthquake event. Therefore it will produce floors forces that are smaller than those of the pESA method of NZS 11709.5:2004.

Secondly, the number of analytical runs that must be undertaken is time consuming.

NZS 1170.5:2002 – Structural design actions – Part 5: Earthquake actions – New Zealand and NZS 1170.5 Supp1:2004 – Structural design actions – Part 5: Earthquake actions – New Zealand Commentary.

The pseudo-Equivalent Static Analysis method (pESA) is now part of NZS 1170.5:2004, through Amendment No.1 (Sept 2016). This method was developed out of the PhD studies of Dr D. Gardiner, University of Canterbury.

A method from NZS 1170.5:2004 and its predecessor, NZS 4203:1992, called "Parts and Components" and "Parts and Portions" respectively, estimates the dynamically amplified inertia forces for each level of a building. These forces are used to design the cladding panels and their connections to the floor. Further, items such as battery stacks or computer banks will be subjected to these amplified accelerations and are designed to resist these accordingly. In the absence of any other "codified" method, designers where applying the "Parts and Components" amplified inertia forces to each level at the same time in order to do the needed 3-dimensinal analysis of a building (to get transfer forces as well as the inertia forces). This approach is very conservative, up to 3.5 times larger than necessary. The pseudo-Equivalent Static method was developed in part as a response to the incorrect, overly conservative use of the "Parts and Components" floor inertias to design floor diaphragms.

PTL and other studies (not included here) have confirmed that the Equivalent Static Analysis of NZS 1170.5, used to design the lateral force resisting elements of buildings, consistently underestimates the forces that develop in diaphragms when compared to those determined by Non-Linear Time History Analysis of the same structures under comparable seismic demands.

6 TWO STUDIES ON THE PEAK FLOOR ACCELERATIONS FOR A RANGE OF BUILDINGS

Reports from the two studies have been appended here,

6.1 PTL – Structural Consultants: "Modelling and analysis of diaphragm behaviour", for UC Quake Centre

The study investigated 7 building types:

A. Four-story residential building Laminated Timber (CLT) with CLT shear walls, with CLT flooring, in Christchurch

B. Three-storey office building reinforced concrete frame, with topped hollowcore flooring, in Christchurch

C. Four-story residential building Laminated Timber (CLT) with CLT shear walls, with CLT flooring, in Wellington

D. Twelve-storey office building reinforced concrete wall-frame hybrid with rib and infill floor, in Wellington

E. Seven-storey residential building reinforced concrete, shear walls, RC frame for gravity and hollowcore floor, in Auckland

F. Eight-storey steel office building, with buckling restrained braces, corrugated steel profile and concrete infill floor, in Christchurch

G. Nine-storey office building structural steel, corrugated steel profile and concrete infill floor, in Auckland

Non-linear time history analysis (NLTHA) was the analysis method. A 2-dimensional model of each structure was developed. Three actual earthquake records were selected for the three sites investigated (Auckland, Wellington, and Christchurch). These sets of records were scaled to meet the 500 year Return Period response spectra from NZS 1170.5:2004.

6.2 Holmes Consulting LP: "Peak Floor Acceleration Study", for the Ministry of Business, Innovation and Employment

The study investigated 3 building types, with 3, 5, 10, and 15 storeys.

The three building types were: reinforced concrete moment frames, reinforced concrete wall buildings, and structural steel moment frames.

A non-linear time history analysis (NLTHA) was employed on a 2-dimensional model of each structure. Three earthquake records were scaled to meet the 500 year Return Period response spectra from NZS 1170.5:2004.

7 SUMMARY OF FINDINGS FROM THE RESEARCH PROGRAMME

Both the studies summarised below were focusing on preliminary assessment of a number of Code and guideline methods for predicting the accelerations of floor diaphragms in buildings.

7.1 PTL – Structural Consultants: "Modelling and analysis of diaphragm behaviour", for UC Quake Centre

The analytical results from the NLTHA have shown that current code-based methods listed in the **Scope of Work**, Item 1., generally underestimate the acceleration response of the structure, except the NZS 1170.5 "Parts and Components" method which over-estimated the numerical acceleration envelopes in most cases.

There were concerns expressed that some of the recorded high peak accelerations would not produce damage in the structure, as the duration of these large accelerations was relatively short. Damage for floor plates, and vertical structures of modern buildings typically arises from displacement of the building and its components. As such, these larger floor accelerations, but of very short duration, would result in displacements so small as to be inconsequential in terms of damage and transfer forces.

PTL investigated using a low pass frequency filter, taking out the high frequencies in the time history analyses. The process was innovative. The amount of filtering was limited at the point where there was no significant difference between the filtered and unfiltered displacement responses of the structures.

The conclusions drawn from the NLTHA by PTL are based on the acceleration response envelopes of the filtered analyses.

The pseudo-Equivalent Static Analysis (pESA), recently included in NZS 1170.5 Supp1:2004, and the Eurocode 8 non-structural elements demand evaluation produced the closest estimates to the NLTHA results. This was in particular for those case study buildings whose response was not significantly influenced by higher frequency response. In other words, the structures that were dominated by the first mode of translator vibration (and the associated displaced shape).

7.2 Holmes Consulting LP: "Peak Floor Acceleration Study", for the Ministry of Business, Innovation and Employment

This study was less comprehensive, less sophisticated than the PTL report. It did however provide confirmation of the trends seen in the PTL outcomes.

Two observations regarding this investigation:

1. The accelerations associated with the higher mode responses found in the NLTHA on the taller structures tend to exceed the floor accelerations of the pESA.

2. There was a concerned expressed (as within the PTL findings) that some of the peak accelerations reported may have lasted a very short time. As such, as it is displacements within the structures that cause damage, the displacements associated with these short duration larger floor accelerations would be small.

The comparison between the pESA envelopes and the NLTHA followed similar trends the unfiltered PTL outcomes. Therefore if a similar filtering of these high, short duration accelerations was employed, as with the PTL study, then a closer match may have been found.

8 QUESTIONS TO BE ANSWERED

Three questions were posed as part of the focusing of the studies:

1. *Can the pseudo-Equivalent Static Analysis method (pESA) can be applied above 9 storeys (a limitation suggested in NZS 1170.5:2004)?*

   The PTL report indicated from one building in the study, that the pESA method might be extended to 12 stories.

   It might be surmised that possibly, independent of height, that the pESA method could be applied provided the dominant displaced shape of the building was that of "first translatory mode".

2. *What is the effect on the magnitude and interstorey distribution of floor forces by applying Peak Ground Acceleration (PGA) up the full height of a structure – a result of using the pESA with flexible structures?*

   In terms of enveloping the peak accelerations up the height of the building, the pESA inverted distribution of overstrength floor forces fits most of the maxima envelopes from the NLTHA (PTL report). Noting that the constant PGA also fits reasonable well with the NLTHA envelopes as well.

   Both the pESA invert distribution of overstrength floor forces and PGA components will err on the safer side (because not all this floor forces will occur at the same time), while not being overly conservative. Noting too, that the pESA distribution is significantly less conservative that methods such a "Parts and Component".

   From these observations we believe that there is nothing to suggest otherwise the using the PGA floor forces up the height of the building where the pESA method recommends these.

3. *How applicable is the pESA method to timber/flexible diaphragms?*

   The study by PTL shows that providing the flexibility of the diaphragm is realistically modelled (as similarly noted by Sabelli et al, and Cowie et al) then the use of the pESA method is reasonable.

9 SUMMARY OF FINDINGS AND RECOMMENDATIONS

- The two studies summarised here have confirmed the applicability of the pseudo-Equivalent Static Analysis (pESA) for estimating the forces that develop in floor diaphragms for seismic design.

- Currently the NZS 1170.5 limitation of 9 storeys for the general use of the pESA is confirmed. There is possibility of applying the pESA to taller structures providing the predominant lateral displaced shape is that of the first translatory mode.

- Certain structures, typically being relatively flexible, the use of floors forces in the pESA equal to the Peak Ground Acceleration (PGA) up the height of those structures is confirmed.

- pESA can be applied to buildings with flexible diaphragms , provided the diaphragm strengths and stiffness are modelled appropriately.

- In Non-Linear Time History Analyse for elements that are displacement sensitive (as against acceleration), the method of filtering out high, short duration accelerations, can produce a better understanding of the force demands of a buildings under seismic loading.

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10 REFERENCES

ASCE 7-10 and ASCE 7-16 – Minimum design loads for buildings and other structures

Cowie, K.A., Fussell, A.J., Clifton, G.C., MacRae, G.A., and Hicks, S.J., 2014, "Seismic Design of composite metal deck and concrete-filled diaphragms – A discussion paper", NZSEE Conference 2014.

Sabelli, R., Sabol, T., and Easterling, W., 2011, "Seismic Design of Composite Streel Deck and Concrete-filled Diaphragms", National Institute of Standards and Technology, NIST GCR 11-917-10.

NZS 1170.5:2002 – Structural design actions – Part 5: Earthquake actions – New Zealand

NZS 1170.5 Supp1:2004 – Structural design actions – Part 5: Earthquake actions – New Zealand Commentary

EN-1998-1:2004 – Eurocode 8 – Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings.

11 APPENDICES

PTL report

HCLP (English, J.) report

Modelling and analysis of diaphragm seismic behaviour

UC Quake Centre

STRUCTURAL TIMBER CONSULTANTS

Date: 13/09/2017
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PTL | Structural Consultants
PO Box 8415
Christchurch 8041
New Zealand

info@ptlnz.com

Prepared by: Francesco Sarti PhD
Structural Engineer

Reviewed by: Tobias Smith PhD ME BE(hons)(Civil) GIPENZ
General manager

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1 GENERAL

PTL have been engaged by the University of Canterbury Quake Centre (UCQC) to perform the nonlinear modelling and analysis of the diaphragm seismic behaviour of a number of case study buildings.

This work has been performed by PTL in collaboration with Des Bull of Holmes Consulting Group, and Robert Finch and Alistair Russel of the University of Canterbury Quake Centre.

The numerical work performed as per scope agreed with the client (UCQC) involved:

- Selection and design of seven buildings representative of current new building design
- Two-dimensional non-linear time-history modelling of the case study buildings
- Interpretation and discussion of non-linear time history results

This report is intended for the client's information and to be shared for technical discussion with competent parties.

PTL shall be informed of any public release of analysis results and reasonable acknowledgement of the contribution of PTL is expected.

2 REFERENCED DOCUMENTS

The following documents have either been used for design of the case study buildings or referenced in the document:

2.1 Standards

ASCE 7-10 – Minimum design loads for buildings and other structures

EN-1998-1:2004 – Eurocode 8 – Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings.

AS/NZS 1170.0:2002 – Structural design actions – Part 0: General principles

AS/NZS 1170.1:2002 – Structural design actions – Part 1: Permanent, imposed and other actions

AS/NZS 1170.2:2002 – Structural design actions – Part 2: Wind actions

NZS 1170.5:2002 – Structural design actions – Part 5: Earthquake actions – New Zealand

NZS 1170.5 Supp1:2004 – Structural design actions – Part 5: Earthquake actions – New Zealand commentary

NZS 3101.1:2006 – Concrete structures standard – Part 1 – The Design of Concrete Structures

NZS 3404:Part 1:1997 – Steel Structures Standard

NZS 3603:1993 – Timber structures standard

2.2 Manufacturer's technical publications

Rothoblaas – Wood Connectors and timber plates. 2015.

Rothoblaas – Seismic-REV – Experimental campaign on Rothoblass products. Mechanical property investigation via monotonic and cyclic loading.

XLam NZ Ltd – Cross Laminated Timber Design Guide, Version V1.4 NZ, March 2013.

2.3 Research literature

Blass, H.J., Fellmoser, P. 2004. Design of solid wood panels with cross layers. *World Conference on Timber Engineering*, Lahti, Finland.

Chiou, B., Darragh, R., Gregor, N., Silva, W. 2008. NGA project strong-motion database. *Earthquake Spectra* 24(1): 23-44.

McKenna, F. 2011. OpenSees: A Framework for Earthquake Engineering Simulation. *Computing in Science and Engg.* 13(4): 58-66.

3 METHODOLOGY

3.1 Lateral load design

The lateral load design of the case study buildings was carried out in accordance to current New Zealand loadings standards and relevant material standards.

Three different design locations were considered for the design of the case study buildings. The summary of the design locations and seismic design parameters are summarized below.

Table 1. Seismic design parameters.

Property	Auckland	Christchurch	Wellington
Soil class	B	D	C
Hazard factor, Z	0.13	0.30	0.40
Return period factor, R	1.0		
Near Fault Factor, N(T,D)	1.0		
Inelastic spectrum reduction factor, k_μ	System dependant (refer to ANNEX A)		

Building-specific seismic design assumptions (i.e. period of vibration, design ductility factor) can be found in ANNEX A.

3.2 Nonlinear models

The nonlinear modelling was performed using the software OpenSEES (McKenna, 2011).

For the different building structural systems, system-specific nonlinear modelling approaches were adopted. The case study buildings can be broken down into three main groups:

- Cross Laminated Timber (CLT) buildings. The lateral load resisting system consisted of CLT wall panels which were connected using a combination of hold-downs and shear brackets. An equivalent truss modelling approach was considered.
- Concrete buildings. The lateral load resisting system for concrete buildings consisted of either reinforced concrete walls, frames or a combination of the two. Potential plastic hinge zones (i.e. beams and column bases) were modelled with fibre element sections representative of the specific reinforcement design, and the capacity-protected elements were modelled as linear elastic elements.
- Steel buildings. The lateral load resisting systems consisted of either Buckling-Restrained Braces (BRBs) or Eccentrically-Braced Frames (EBFs). For both these building typologies nonlinear fibre elements were used in the analysis.

ANNEX A reports more detailed information on the nonlinear model development of the case study buildings.

3.3 Ground motion selection and scaling

Three sets of earthquake records were selected from the PEER NGA West2 database (Chiou *et al.*, 2008) to represent the design assumptions as stated in Section 3.1 and in accordance to Clause 5.5 of NZS 1170.5.Tables 2 to 4 summarize the selected record sets for each design location (i.e. Auckland, Christchurch and Wellington) and Figures 1 to 3 show a comparison of the target spectrum versus the average spectra of the record sets scaled to NZS 1170.5 at a period of 1.0 s.

Table 2. Auckland record set.

ID	PEER ID	Earthquake Name	Year	Station Name	M_w
1	1	Helena_ Montana-01	1935	Carroll College	6.0
2	17	Southern Calif	1952	San Luis Obispo	6.0
3	28	Parkfield	1966	Cholame - Shandon Array #12	6.2
4	40	Borrego Mtn	1968	San Onofre - So Cal Edison	6.6
5	146	Coyote Lake	1979	Gilroy Array #1	5.7
6	216	Livermore-01	1980	Tracy - Sewage Treatm Plant	5.8
7	233	Mammoth Lakes-02	1980	Convict Creek	5.7
8	236	Mammoth Lakes-03	1980	Convict Creek	5.9
9	238	Mammoth Lakes-03	1980	Long Valley Dam (L Abut)	5.9
10	551	Chalfant Valley-02	1986	Convict Creek	6.2

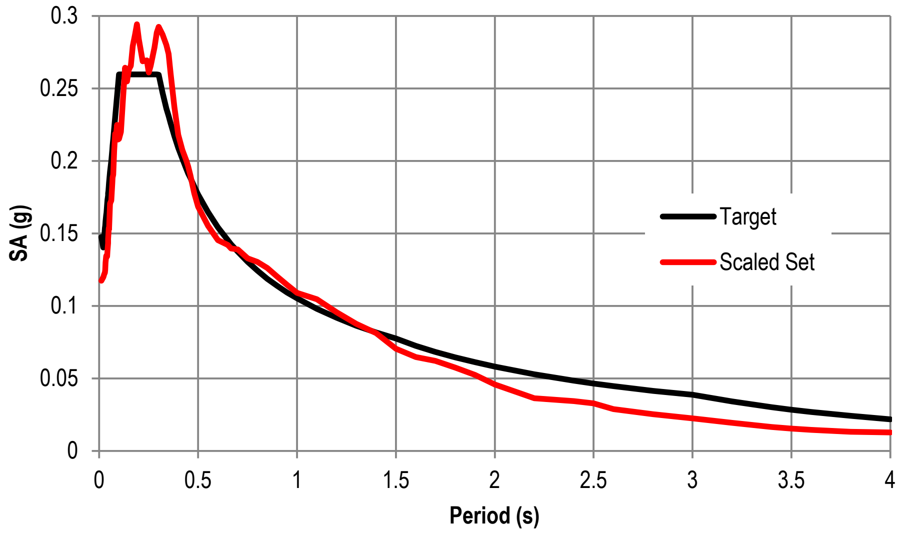


Figure 1. Auckland record set average spectrum (scaled at T_1 = 1.0s).

Table 3. Christchurch record set.

ID	PEER ID	Earthquake Name	Year	Station Name	M_w
1	6	Imperial Valley-02	1940	El Centro Array #9	7.0
2	26	Hollister-01	1961	Hollister City Hall	5.6
3	175	Imperial Valley-06	1979	El Centro Array #12	6.5
4	721	Superstition Hills-02	1987	El Centro Imp. Co. Cent	6.5
5	729	Superstition Hills-02	1987	Imperial Valley Wildlife Liquefaction Array	6.5
6	761	Loma Prieta	1989	Fremont - Emerson Court	6.9
7	850	Landers	1992	Desert Hot Springs	7.3
8	900	Landers	1992	Yermo Fire Station	7.3
9	1258	Chi-Chi_ Taiwan	1999	HWA005	7.6
10	1762	Hector Mine	1999	Amboy	7.1

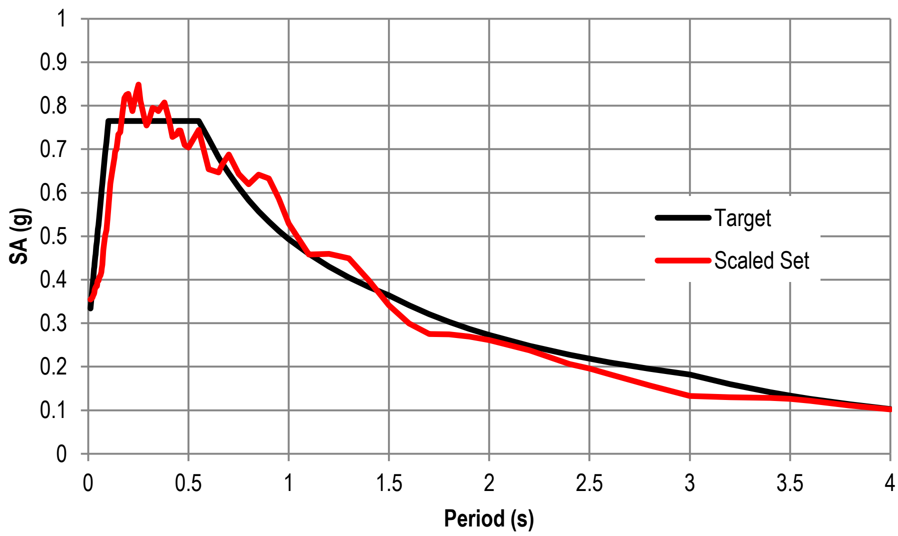


Figure 2. Christchurch record set average spectrum (scaled at T_1 = 1.0s).

Table 4. Wellington record set.

ID	PEER ID	Earthquake Name	Year	Station Name	M_w
1	161	Imperial Valley-06	1979	Brawley Airport	6.5
2	174	Imperial Valley-06	1979	El Centro Array #11	6.5
3	175	Imperial Valley-06	1979	El Centro Array #12	6.5
4	179	Imperial Valley-06	1979	El Centro Array #4	6.5
5	183	Imperial Valley-06	1979	El Centro Array #8	6.5
6	725	Superstition Hills-02	1987	Poe Road (temp)	6.5
7	882	Landers	1992	North Palm Springs	7.3
8	1209	Chi-Chi_ Taiwan	1999	CHY047	7.6
9	1484	Chi-Chi_ Taiwan	1999	TCU042	7.6
10	1491	Chi-Chi_ Taiwan	1999	TCU051	7.6

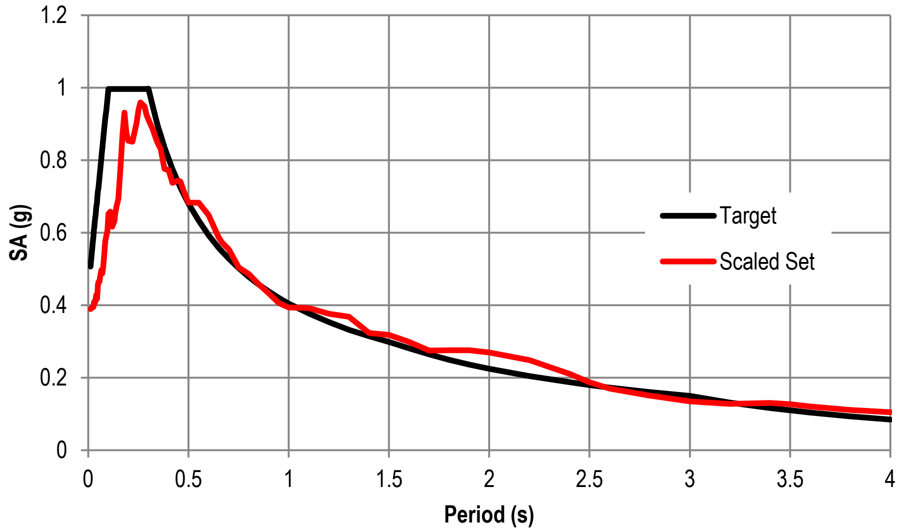


Figure 3. Wellington record set average spectrum (scaled at T_1 = 1.0s).

3.4 Nonlinear Time-History Analysis (NLTHA) and results

Nonlinear time-history analyses were performed in accordance with Clause 6.4 of NZS 1170.5 and with the record sets presented in Section 3.3.

The results are presented in the form of shear, moment, displacement, drift and diaphragm acceleration envelopes in Figures 4 to 10.

Floor acceleration envelopes are then compared to current standard-based design methodology and in particular:

- Equivalent Static Method (ESA) in accordance to Clause 6.2 of NZS 1170.5
- Pseudo Equivalent Static Analysis (pESA) in accordance to Clause C5.7.A2.3 of NZS 1170.5 Supp 1
- Part and components method (P+C) in accordance to Section 8 of NZS 1170.5
- Diaphragm forces in accordance (ASCE 7-10) to Section 12.10.1.1 of ASCE 7-10
- Non-structural elements demand evaluation (EC8) in accordance to Section 4.3.5 of EN 1998 (Eurocode 8)

The results are also compared to the short period spectral acceleration for the building design location (referred to as MSa in the figures below).

4 Results

4.1 Building A

Building A was a four-storey Cross Laminated Timber (CLT) residential building located in Christchurch. The building's lateral load resisting system consisted of CLT walls which provide shear and moment capacity through steel connectors (hold-downs and brackets). The gravity load resisting system also consisted of CLT flooring which transfers the vertical loads to the supporting CLT wall structure.

Figure 4 shows the diaphragm acceleration envelopes resulting from NLTHA compared with the different available design methods.

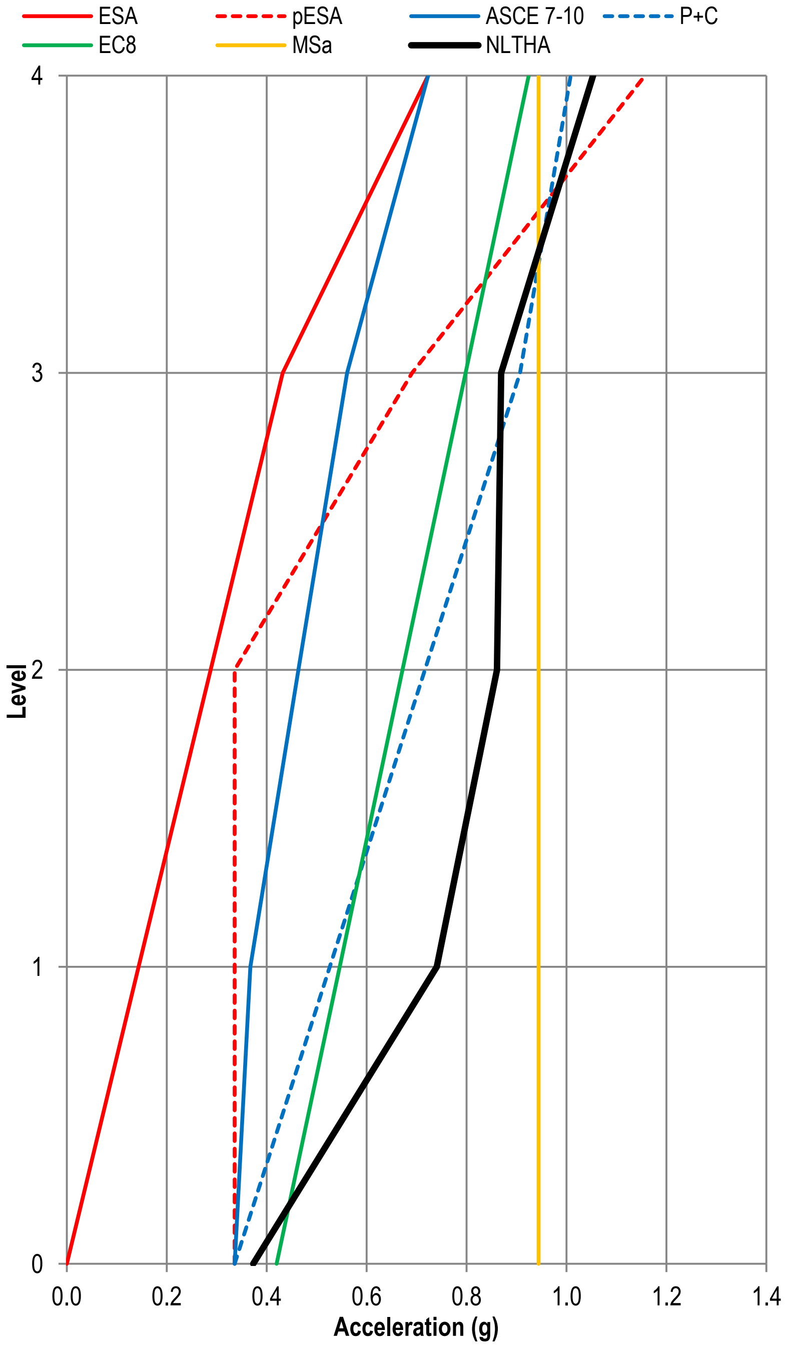


Figure 4. Building A. Diaphragm acceleration envelopes.

More details about the analysis methodology as well as the shear, moment, displacement and drift envelopes are reported in Section A.1.

4.2 Building B

Building B was a three-storey concrete office structure located in Christchurch. Lateral loads were resisted by reinforced concrete frames which also carried gravity loads and supported a hollow core flooring system.

Figure 5 shows the diaphragm acceleration envelopes of Building B compared to current code-based prediction equations.

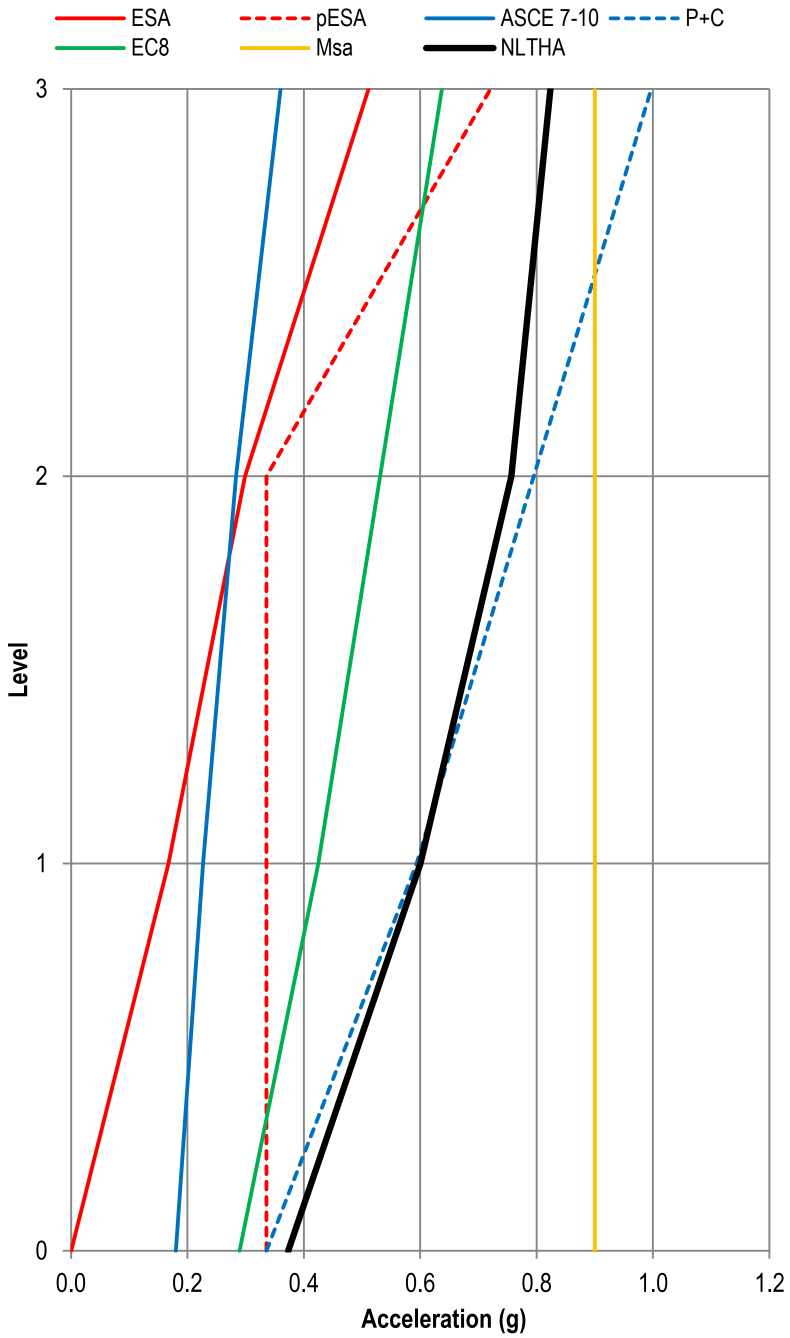


Figure 5. Building B. Diaphragm acceleration envelopes.

More details about the design and analysis methodology as well as the shear, moment, displacement and drift envelopes are reported in Section A.2.

4.3 Building C

Building C was a residential building located in Wellington. The lateral load resisting system of the structure was a series of Cross Laminated Timber shear walls in the two directions of the building. The walls were also carrying the vertical actions which were transferred to the panels through CLT flooring.

The diaphragm acceleration envelopes resulting from NLTHA are shown in Figure 6a and these are compared to code-based design approaches as outlined in Section 3.4.

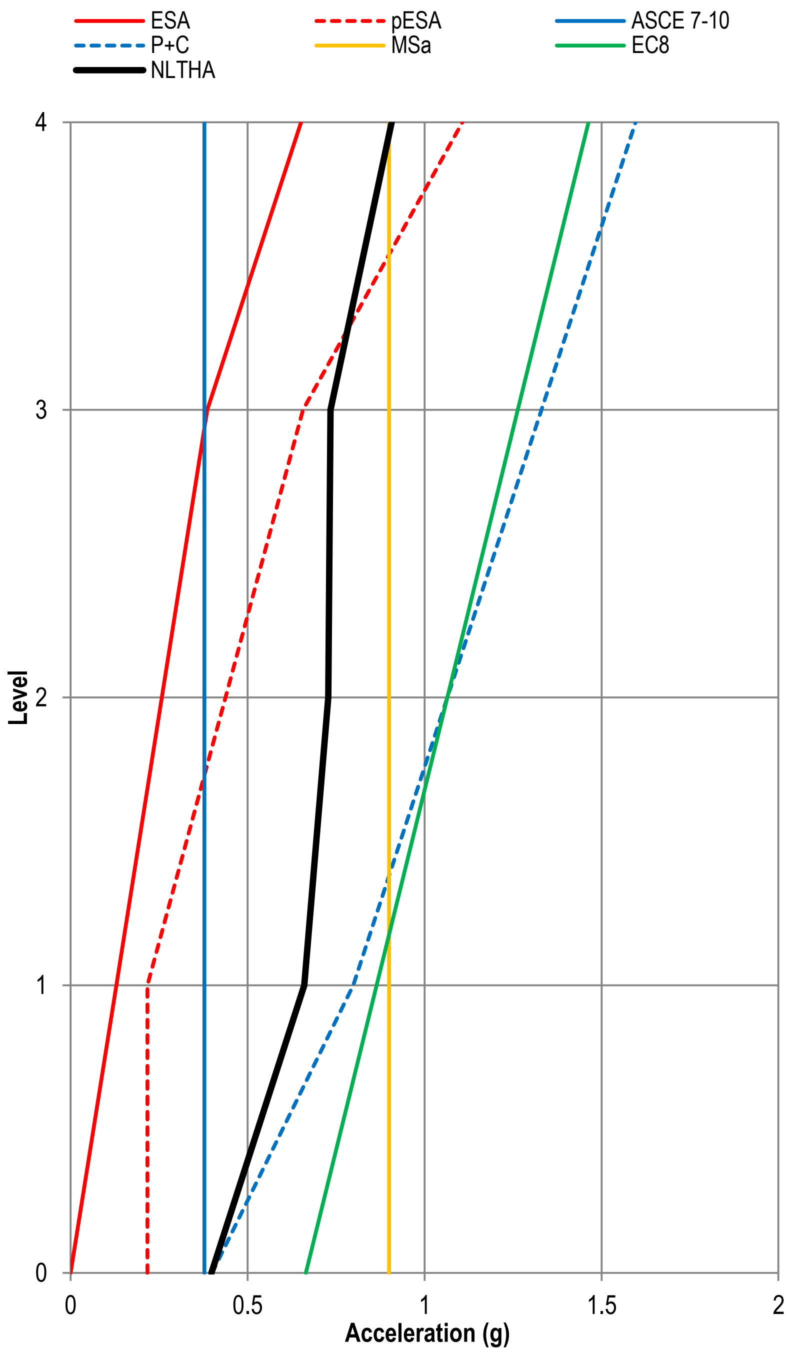


Figure 6. Building C. Diaphragm acceleration envelopes.

The analysis methodology of Building C is discussed in more detail in Section A.3.

4.4 Building D

Building D was a twelve-storey office building with residential penthouse at the top level of the building and it was located in Wellington. A hybrid concrete wall-frame system provided the lateral stability to the building and partly carries the vertical loads which were transferred to the concrete frame through a rib and infill flooring system.

The diaphragm acceleration envelopes resulting from the numerical analyses are reported in Figure 7.

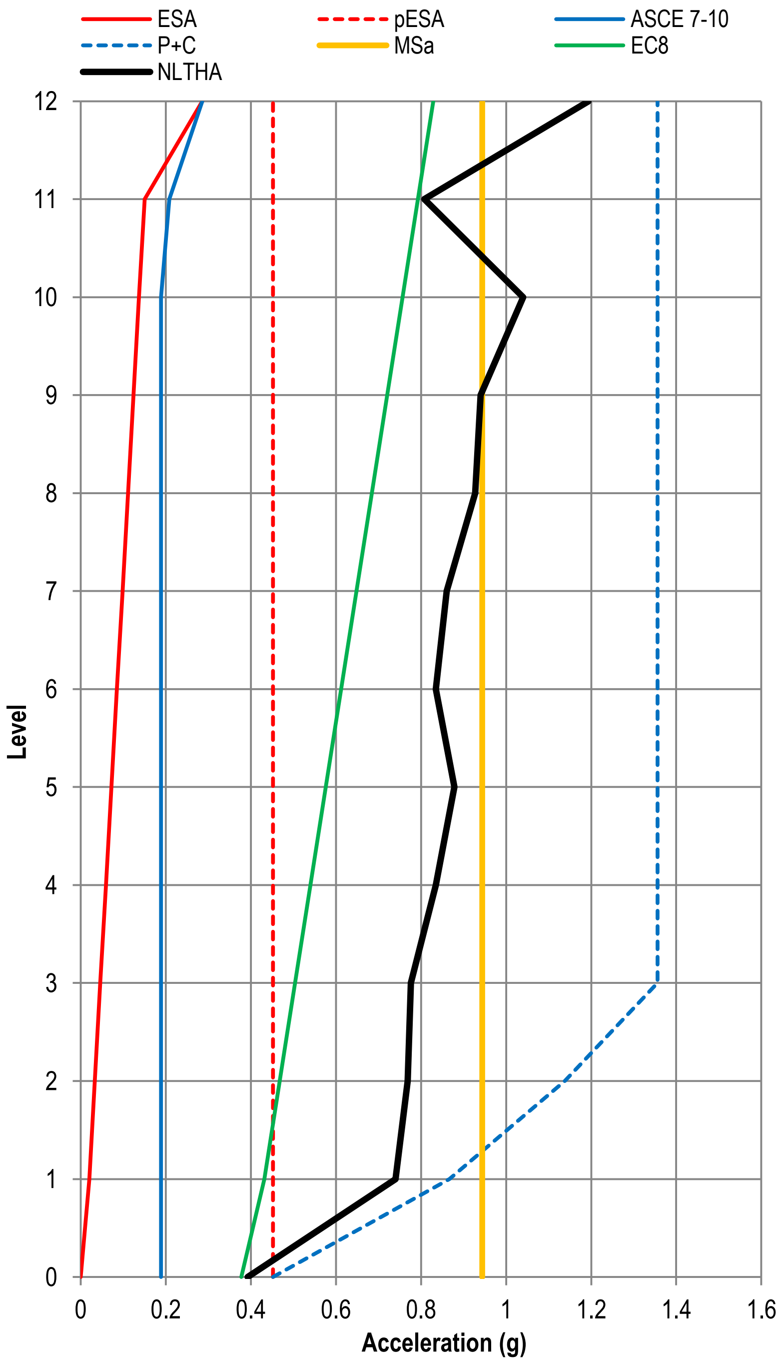


Figure 7. Building D. Diaphragm acceleration envelopes.

A more detailed discussion on the design and modelling methodology for Building D is shown in Section A.4.

4.5 Building E

Building E was a seven-storey concrete residential building located in Auckland. Two C-shaped concrete walls provided the lateral load stability to the structure, and precast hollow core flooring and concrete frames carried the vertical loads.

The diaphragm acceleration envelopes are shown in Figure 8a and the filtered results in Figure 8b.

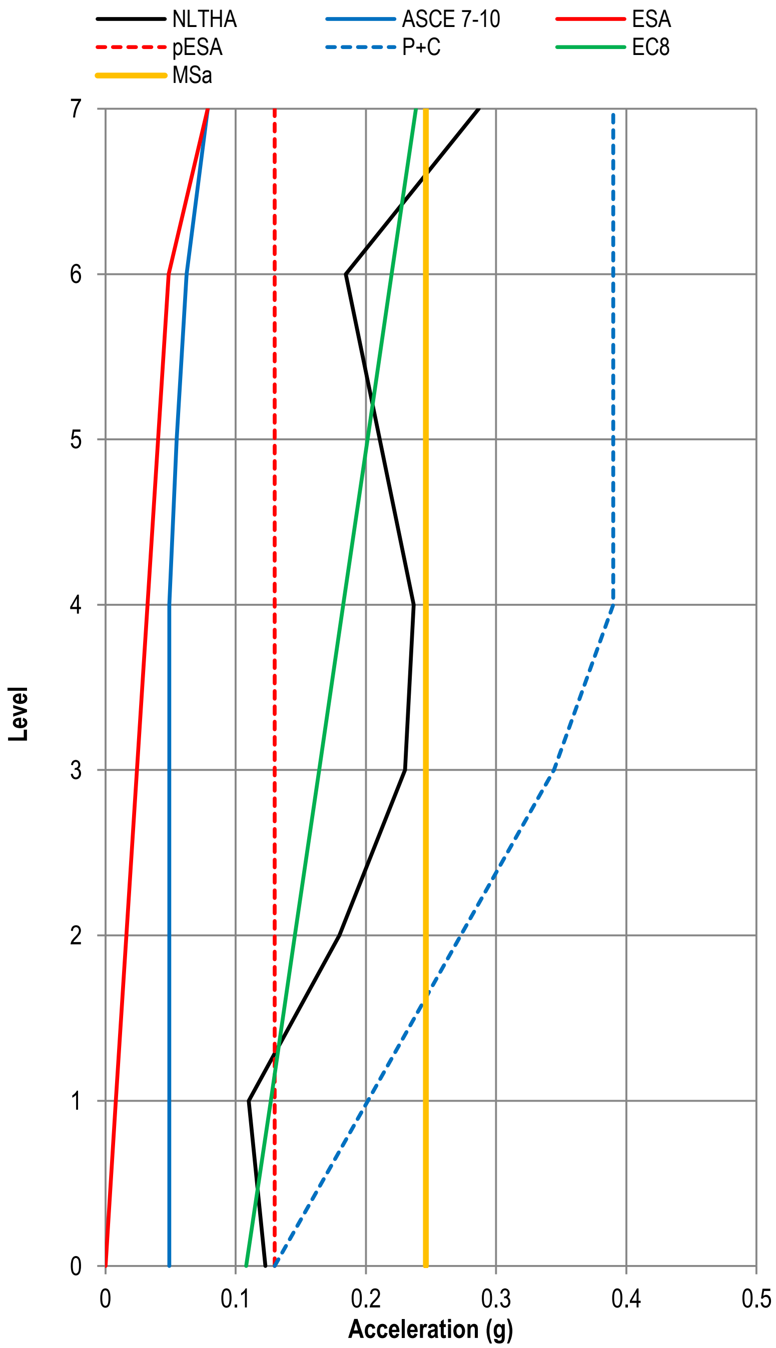


Figure 8. Building E. Diaphragm acceleration envelopes.

Section A.5 provides a more detailed overview of the design parameters as well as the modelling methodology for this building.

4.6 Building F

Building F was an eight-storey steel office building located in Christchurch. Buckling-Restrained Braces (BRBs) were the main lateral load resisting system. A structural steel frame supporting corrugated metal and concrete flooring provided vertical load capacity.

Figure 9 shows the acceleration envelopes and filtered envelopes for Building F.

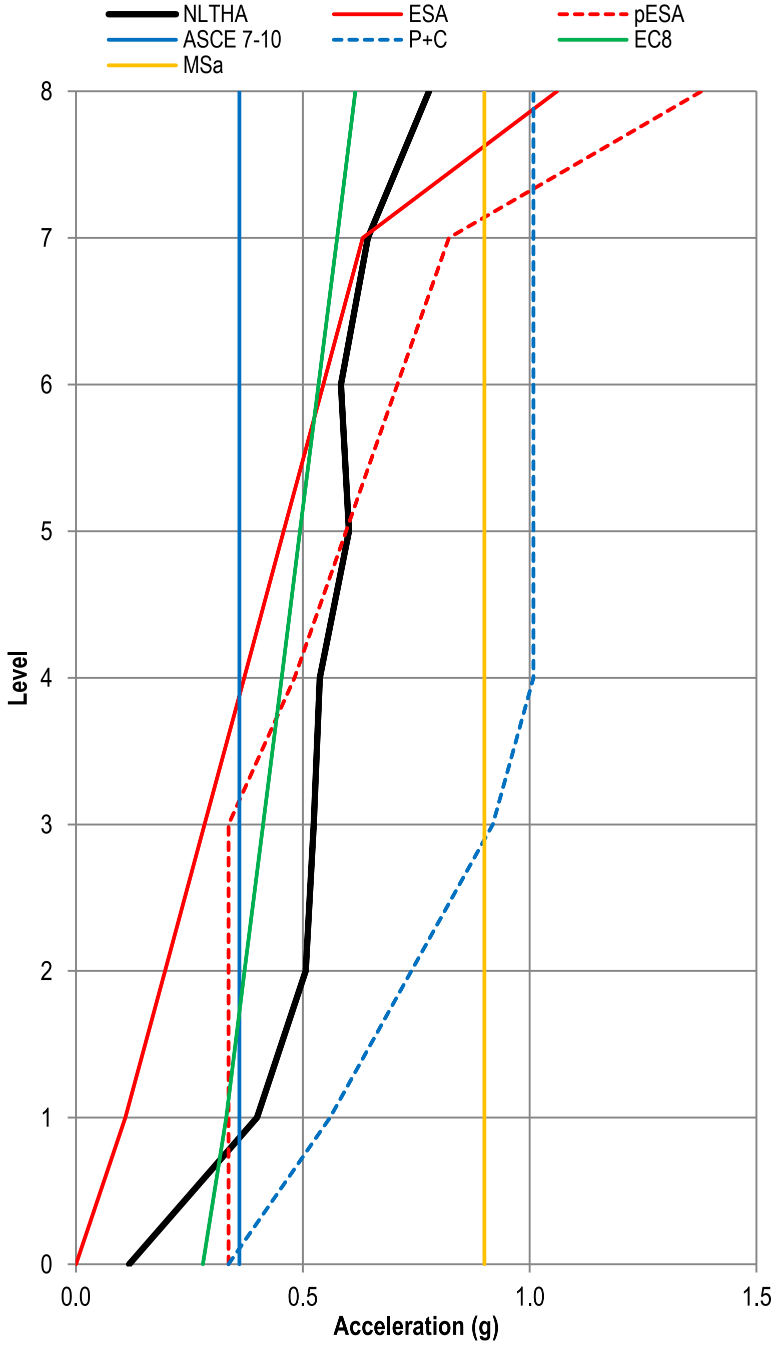


Figure 9. Building F. Diaphragm acceleration envelopes.

Section A.6 reports additional information on the design and model development of building F.

4.7 Building G

Building G was a nine-storey office steel building located in Auckland. The lateral load resisting system consisted of Eccentrically-Braced Frames (EBFs) and the vertical loads were carried through a corrugated steel-concrete floor supported by a steel frame structure.

The NLTHA results are plotted in Figure 10a and compared to code-based prediction equations.

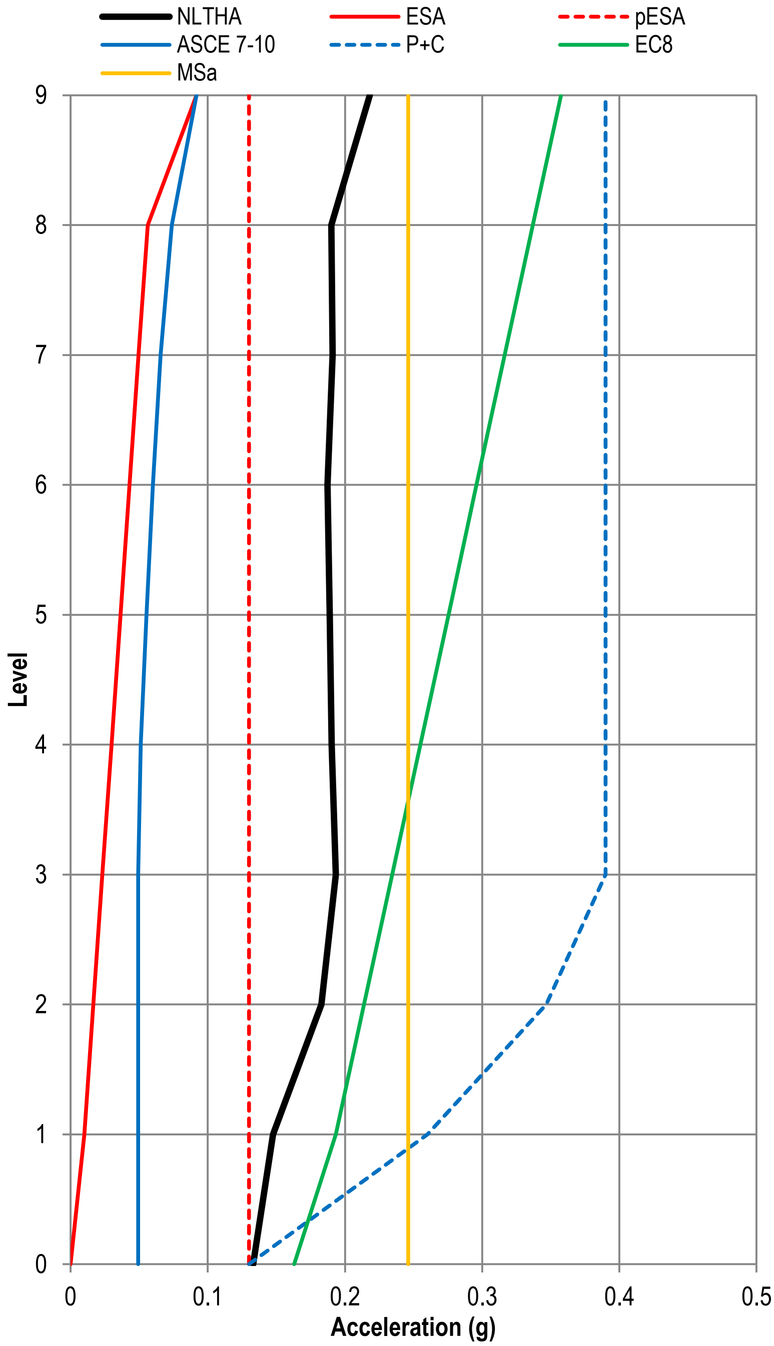


Figure 10. Building G. Diaphragm acceleration (a) envelopes and (b) low-pass filtered envelopes.

Further discussion on the design and modelling methodology can be found in Section A.7.

5 FILTERED RESULTS

The analysis of the diaphragm acceleration time-histories resulting from the numerical analyses have shown that acceleration peaks had a short duration in most cases, suggesting that maximum accelerations were influenced by high-frequency energy content.

While these peaks do occur, it can be arguable that they are capable of causing significant damage to the structure as damage is more closely related to displacement rather than force. Non-linear time-history analysis results have shown that while high-frequency acceleration peaks were observed (see Figure 11a, dashed black line), no significant high-frequency peak in the displacement response was notable (see Figure 11b).

In an attempt to remove high-frequency energy content from the acceleration response of the case study buildings, the acceleration time-histories resulting from the numerical analyses were post-processed using a low-pass filter.

The low-pass filter was applied to the set of numerical results by filtering energy content from frequencies higher than a selected cut-off value. This cut-off frequency was set for each case study building in order to include a total modal participating mass of at least 90% in accordance to the modal analysis results shown in the following sections. This assumption is consistent with code-based approaches for response spectrum analysis as per Clauses 6.3.3 and C6.3.3 of NZS 1170.5.

An example of the results of the low-pass filter is shown in Figure 11. It is also notable that the acceleration response is significantly influenced by higher mode effects (second and third mode), while the displacement response of Building D is mostly governed by first mode response with a small influence of the second mode of vibration.

As the low-pass filter of these results is applied, the acceleration response displays reduced peaks confirming that high-frequency content has been filtered, while the displacement response does not show significant difference from unfiltered results.

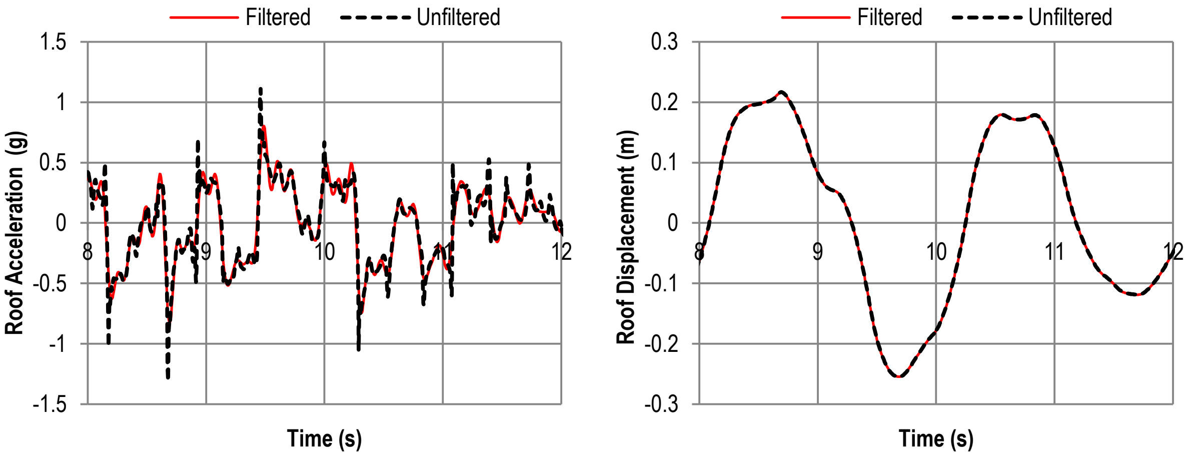


Figure 11. Building D, EQ01 (a) acceleration and (b) displacement response (filtered and unfiltered).

5.1 Building A

The modal analysis results for Building A as well as the diaphragm acceleration low-pass filtered envelopes are presented below.

Table 5. Building A. Modal analysis results.

No.	T	f	Part. Mass		Cum. Part. Mass	
	(s)	(Hz)	(t)	(%)	(t)	(%)
1	0.32	3.2	21	77.6	21	77.6
2	0.12	8.5	5	17.8	26	95.4
3	0.07	13.7	1	3.7	27	99.2
4	0.06	17.4	0	0.8	27	100.0

First and second modes of vibration include 95.4% of the cumulative participating mass of Building A; therefore, the low-pass frequency cut was set at 8.5Hz. The resulting acceleration envelope compared to code-based prediction methods is shown in Figure 12.

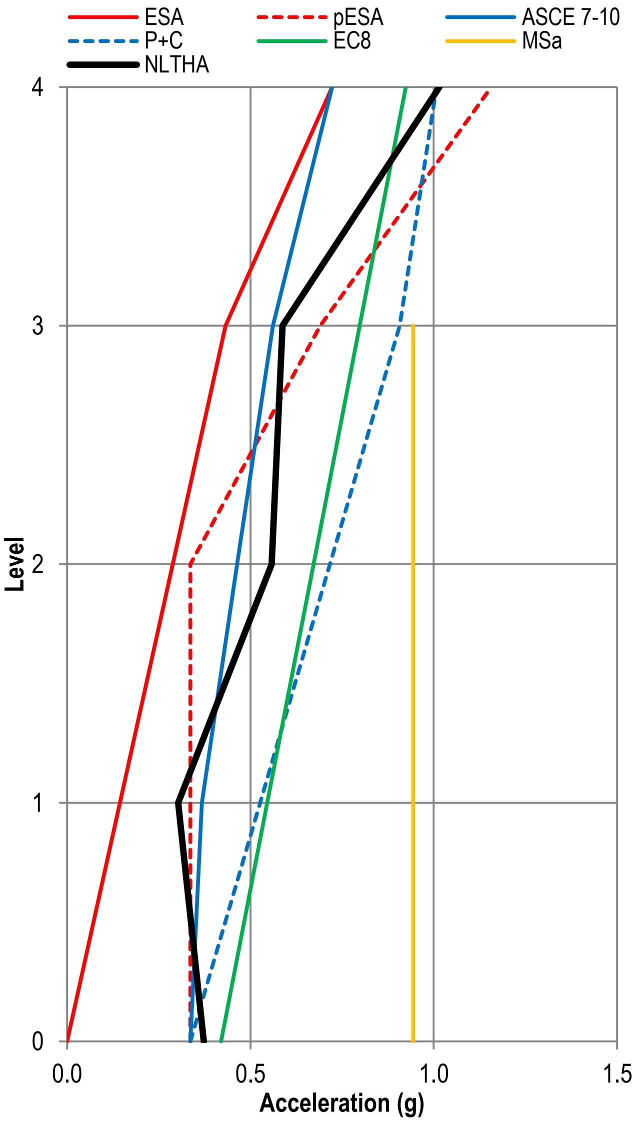


Figure 12. Building A. Diaphragm acceleration low-pass filtered envelopes.

5.2 Building B

Building B the modal analysis results are listed in Table 6.

Table 6. Building B. Modal analysis results.

No.	T	f	Part. Mass		Cum. Part. Mass	
	(s)	(Hz)	(t)	(%)	(t)	(%)
1	0.47	2.1	258	86.3	258	86.3
2	0.13	7.8	35	11.7	293	98.0
3	0.06	17.6	6	2.0	299	100.0

Figure 13 shows the diaphragm acceleration low-pass filtered envelopes of Building B compared to current code-based prediction equations. The low-pass filtered results were evaluated considering a pass frequency of 8 Hz which resulted in the filtering of energy contributions at frequencies higher than the one of the second mode of vibration of the structure.

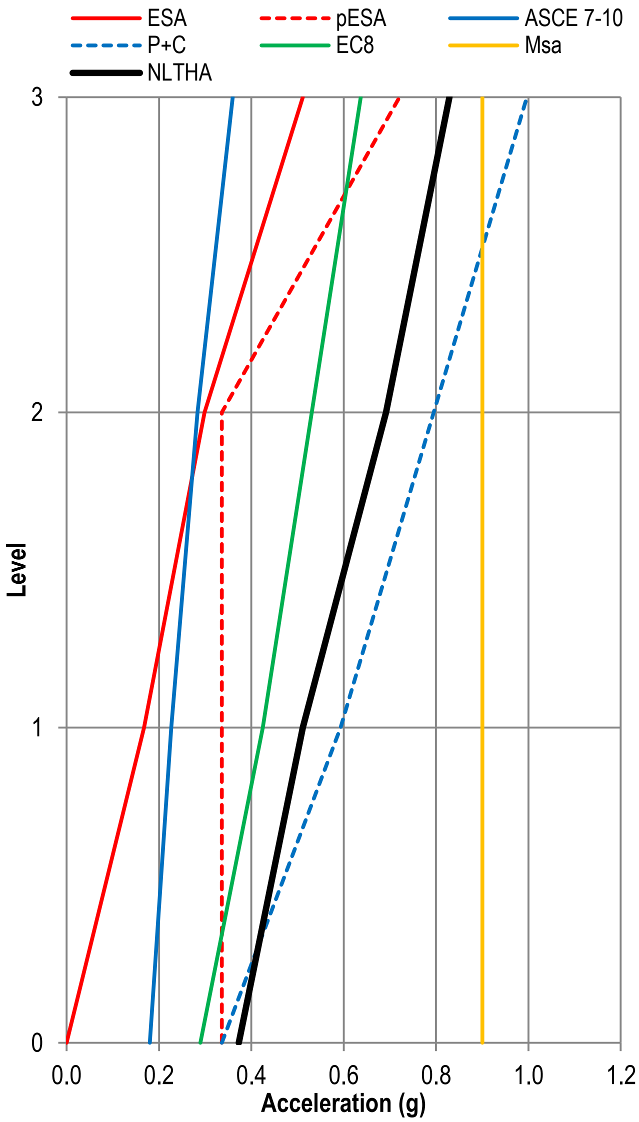


Figure 13. Building B. Diaphragm acceleration low-pass filtered envelopes.

5.3 Building C

The modal properties of Building C are reported in the table below.

Table 7. Building C. Modal analysis results.

No.	T (s)	f (Hz)	Part. Mass (t)	Part. Mass (%)	Cum. Part. Mass (t)	Cum. Part. Mass (%)
1	0.30	3.4	11	77.5	11	77.5
2	0.12	8.3	2	16.1	13	93.6
3	0.08	12.6	1	4.6	14	98.2
4	0.06	17.0	0	1.8	14	100.0

The diaphragm acceleration filtered envelopes resulting from NLTHA are shown in Figure 14. The low-pass frequency cut-off was set at 8.5 Hz.

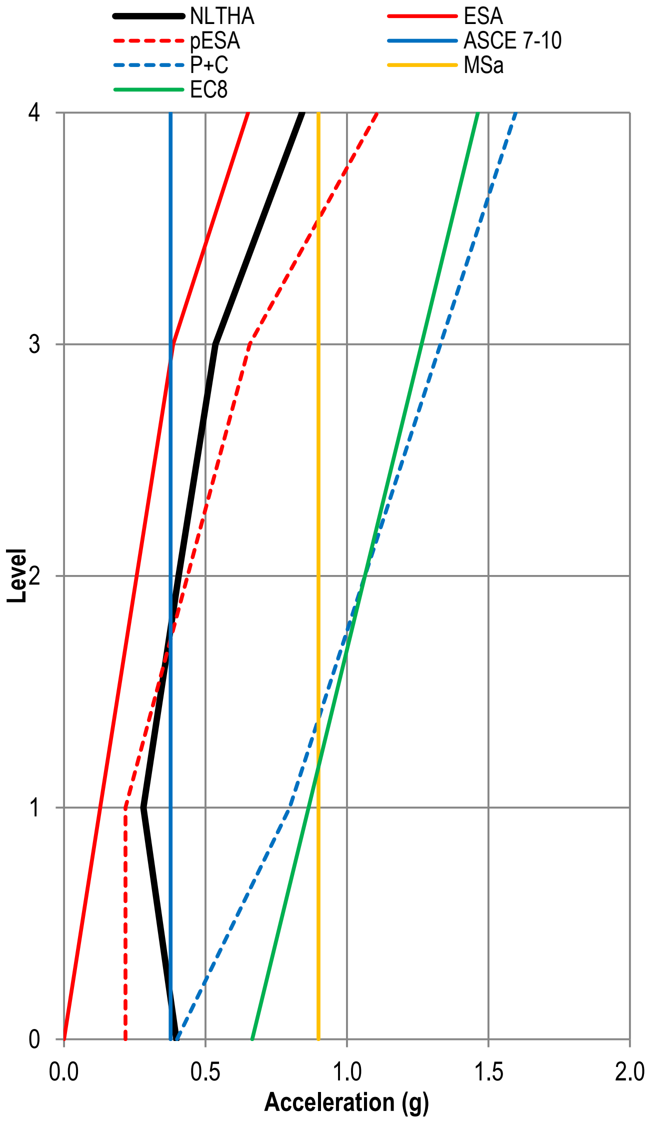


Figure 14. Building C. Diaphragm acceleration low-pass filtered envelopes.

5.4 Building D

The modal properties of Building D are reported in Table 8.

Table 8. Building D. Modal analysis results.

No.	T (s)	f (Hz)	Part. Mass (t)	Part. Mass (%)	Cum. Part. Mass (t)	Cum. Part. Mass (%)	No.	T (s)	f (Hz)	Part. Mass (t)	Part. Mass (%)	Cum. Part. Mass (t)	Cum. Part. Mass (%)
1	1.446	0.7	2191	65.5	2191	65.5	7	0.017	57.9	22	0.7	3327	99.5
2	0.289	3.5	605	18.1	2795	83.6	8	0.013	77.0	10	0.3	3337	99.8
3	0.110	9.1	243	7.3	3038	90.8	9	0.010	98.5	5	0.1	3342	99.9
4	0.057	17.4	138	4.1	3176	95.0	10	0.008	120.9	2	0.1	3344	100.0
5	0.035	28.3	83	2.5	3259	97.4	11	0.007	141.5	1	0.0	3345	100.0
6	0.024	41.7	46	1.4	3305	98.8	12	0.006	156.6	0	0.0	3345	100.0

The diaphragm acceleration envelopes resulting from the numerical analyses are reported in Figure 15. The results were low-pass filtered at 9.5Hz.

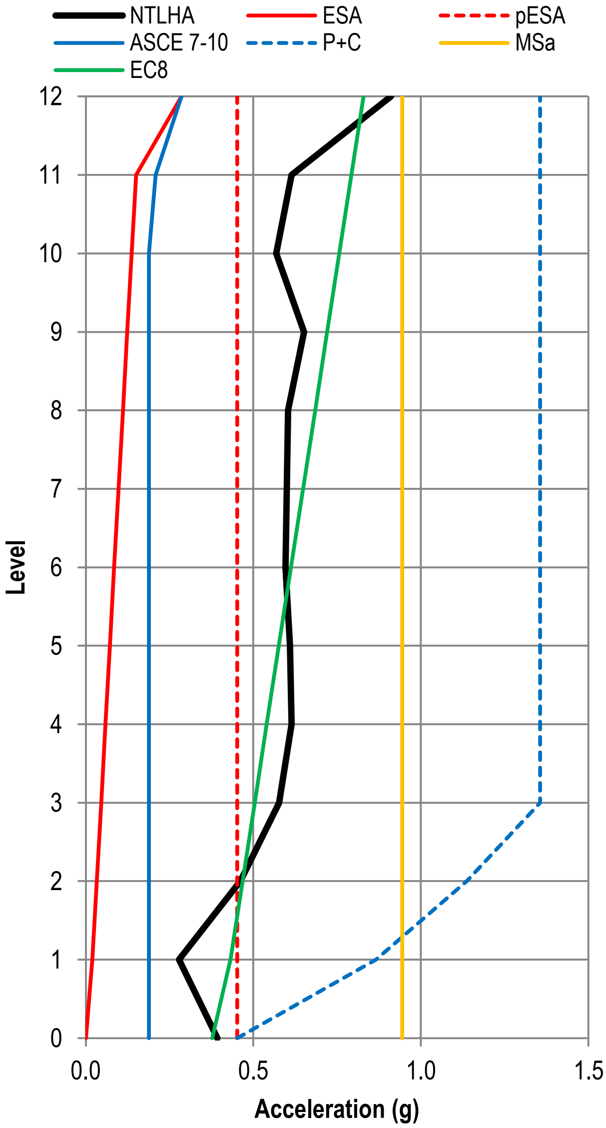


Figure 15. Building D. Diaphragm acceleration low-pass filtered envelopes.

5.5 Building E

The modal analysis results for Building E are shown in Table 9.

Table 9. Building E. Modal analysis results.

No.	T	f	Part. Mass		Cum. Part. Mass	
	(s)	(Hz)	(t)	(%)	(t)	(%)
1	0.32	3.1	1767	64.7	1767	64.7
2	0.05	19.3	555	20.3	2322	85.0
3	0.02	53.9	202	7.4	2524	92.4
4	0.01	104.8	108	3.9	2632	96.3
5	0.01	170.0	61	2.2	2693	98.6
6	0.00	243.0	30	1.1	2724	99.7
7	0.00	305.5	9	0.3	2733	100.0

The diaphragm acceleration filtered envelopes are shown in Figure 16. The low-pass filtered acceleration had a pass frequency of 55Hz (above the third mode frequency).

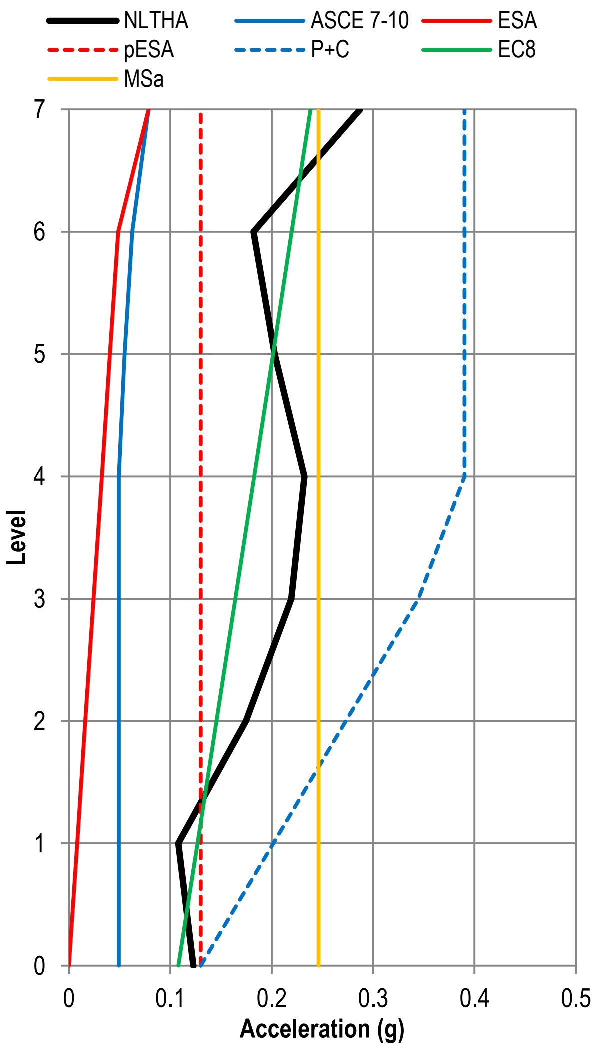


Figure 16. Building E. Diaphragm acceleration (a) envelopes and (b) low-pass filtered envelopes.

5.6 Building F

The modal properties of Building F are reported in Table 10.

Table 10. Building F. Modal analysis results.

No.	T (s)	f (Hz)	Part. Mass (t)	Part. Mass (%)	Cum. Part. Mass (t)	Cum. Part. Mass (%)	No.	T (s)	f (Hz)	Part. Mass (t)	Part. Mass (%)	Cum. Part. Mass (t)	Cum. Part. Mass (%)
1	0.98	1.0	238	73.9	238	73.9	5	0.12	8.1	2	0.6	320	99.3
2	0.33	3.0	59	18.4	297	92.3	6	0.11	9.1	1	0.5	321	99.8
3	0.19	5.1	16	4.9	313	97.2	7	0.10	10.2	0	0.1	322	99.9
4	0.14	7.0	5	1.5	318	98.7	8	0.09	11.6	0	0.1	322	100.0

Figure 17 shows the acceleration low-pass filtered envelopes for Building F. A low-pass filter at 5.5Hz was used to filter the acceleration results shown below.

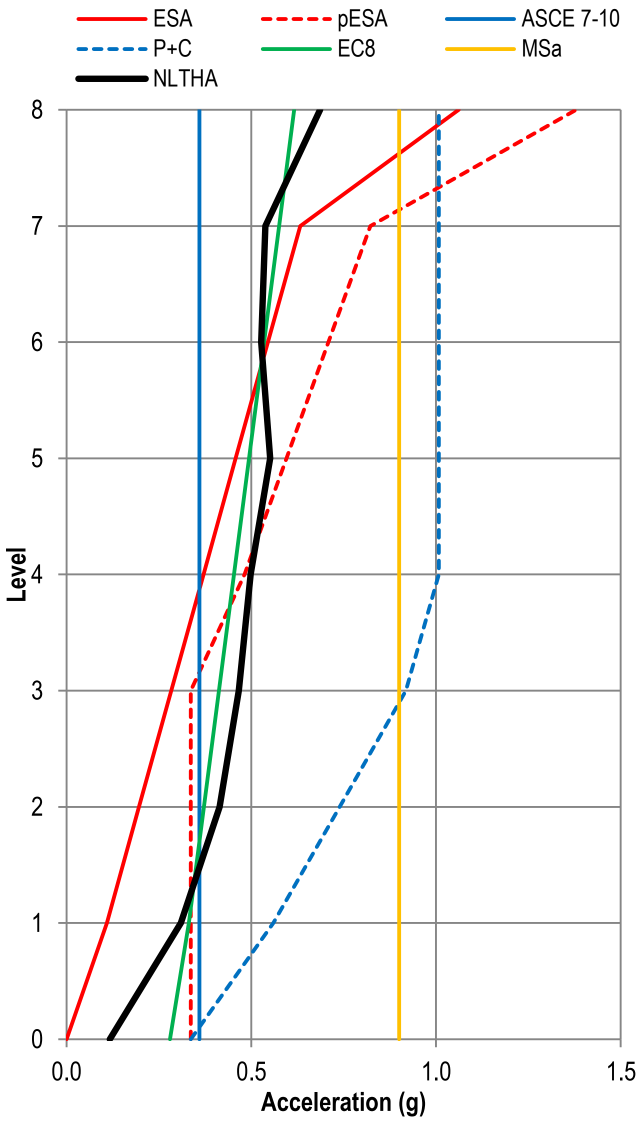


Figure 17. Building F. Diaphragm acceleration low-pass filtered envelopes.

5.7 Building G

The modal properties of the structure are reported in the following table.

Table 11. Building F. Modal analysis results.

No.	T	f	Part. Mass		Cum. Part. Mass		No.	T	f	Part. Mass		Cum. Part. Mass	
	(s)	(Hz)	(t)	(%)	(t)	(%)	(Hz)	(t)	(%)	(t)	(%)	(t)	(%)
1	1.792	0.6	954	70.4	954	70.4	6	0.129	7.7	1	0.1	1355	99.9
2	0.547	1.8	311	22.9	1265	93.3	7	0.115	8.7	1	0.1	1356	99.9
3	0.286	3.5	69	5.1	1334	98.4	8	0.098	10.2	0	0.0	1356	100.0
4	0.199	5.0	14	1.0	1348	99.4	9	0.087	11.5	0	0.0	1356	100.0
5	0.154	6.5	5	0.4	1353	99.7							

Figure 18 shows the acceleration results which were low-passed filtered at 3.5Hz.

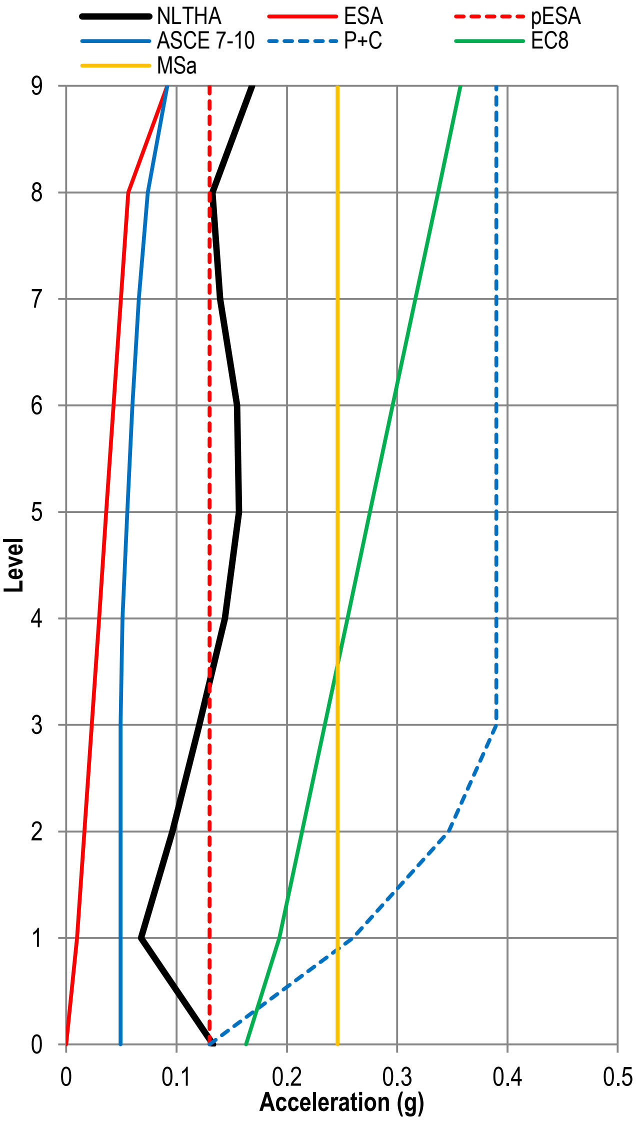


Figure 18. Building G. Diaphragm acceleration low-pass filtered envelopes.

6 DISCUSSION

Non-linear time-history analyses were performed on seven case study buildings designed for lateral load resistance to NZS1170.5 and relevant material standards. The buildings considered were either for residential or office use and were located in Auckland, Wellington or Christchurch.

Dynamic analysis results were output in terms of shear, moment, diaphragm acceleration, displacement and drift envelopes. In particular, the focus of this work was the investigation of diaphragm acceleration envelopes and the comparison of these to current code-based methods.

In all structures, in all considered zones, diaphragm forces were underestimated across most floors by the equivalent static analysis forces.

6.1 Evaluation of code based methods

The evaluation of code based methods showed that:

- The numerical results have shown that current code-based methods generally underestimate the acceleration response of the structure, except NZS 1170.5 parts and components method which over-estimated the numerical acceleration envelopes in most cases.
- The pseudo-Equivalent Static Analysis (pESA) recently included in NZS 1170.5 Supp 1 and the Eurocode 8 non-structural elements demand evaluation produced the closest estimates, in particular for those case study buildings whose response was not significantly influenced by higher frequency response.

6.2 Application of low pass filter

During the data analysis process of the NLTHA results low-pass filtering of the acceleration results was performed. This was done to filter high frequency response contributions and thus eliminating high-frequency energy content from the building's response. It is suggested that this may be a suitable way of rationalising NLTHA output as although peaks do occur in diaphragm forces it is debatable whether or not very high frequency peaks have significant damage potential.

As expected, filtered acceleration envelopes displayed lower values than unfiltered results except for buildings not significantly affected by higher mode effects.

The comparison of filtered acceleration results to code-based methods suggested that pESA and EC8 method provided a good estimation of the floor acceleration (or diaphragm forces) of the case study buildings.

DISCLAIMER

(1) The numerical study presented in this report was aimed at a preliminary assessment of suitability of current code-based approaches in the prediction of diaphragm accelerations. As this study was limited to a small number of case study buildings, a limited number of parameters influencing the dynamic response of a building were considered. Although clear trends did emerge, it is therefore suggested that further investigations are carried out.

(2) Low-pass filtering was performed on acceleration results as outlined in this document to reduce the high-frequency energy contributions which were significantly affecting the acceleration response of the case study buildings. This was done due to the theory that these high frequency contributions have low damage potential however, the applicability and correctness of the post-processing of numerical data based on low-pass filtering requires more in-depth study to be performed.

ANNEX A ANALYSIS METHODOLOGY

A.1 Building A

A.1.1 Lateral load design

Building A is a four-storey residential CLT building located in Christchurch. The plan view and elevation of the building is shown in Figure 19.

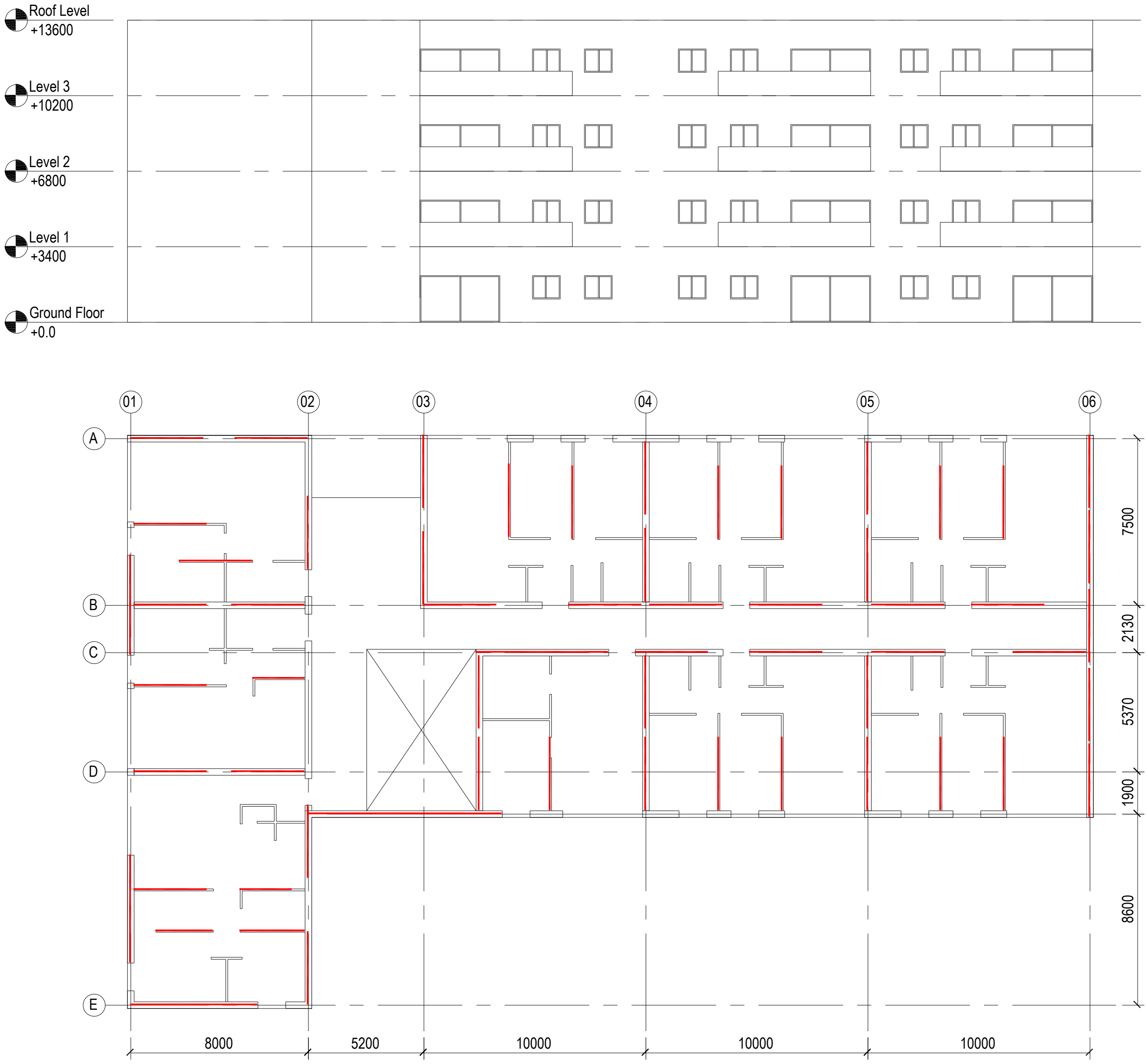


Figure 19. Building A. Plan view and elevation.

The lateral load resisting system of this building consists of Cross Laminated Timber walls connected using a combination of hold-down connectors and brackets which provide the moment and shear capacity to the system, respectively. The plan view in Figure 19 highlights in red the location of the shear walls considered for the lateral load design of the building. The shear walls had a standard length of 3.3m.

A summary of the lateral load design parameters is provided in the list below:

- Soil type D
- Hazard factor, Z = 0.30

- Return period factor, R = 1.0
- Near fault factor, N(T,D) = 1.0
- Period of vibration, T_1 = 0.50s
- Structural ductility factor, μ = 2.0
- Structural performance factor, S_p = 0.7
- Inelastic spectrum scaling factor, k_μ = 1.71

The results of the equivalent static method in accordance to NZS 1170.5 are reported in Table 12.

Table 12. Building A. Equivalent static method results.

Level	H_i *(m)*	h_i *(m)*	W_i *(kN)*	W_ih_i *(m·kN)*	F_i/V *(-)*	V_i/V *(-)*	F_i *(kN)*	V_i *(kN)*	M_i *(kNm)*
1	3.4	3.4	2363	8034	0.118	1.000	340	2891	29117
2	3.4	6.8	1967	13376	0.196	0.882	566	2551	19286
3	3.4	10.2	1967	20063	0.294	0.687	849	1985	10611
4	3.4	13.6	1571	21366	0.393	0.393	1136	1136	3862
		Σ	7868	62838.8	1.000		2891		

For the purpose of this numerical analysis one of the walls on gridline 04 was considered for design and analysis. In accordance to the equivalent static method and assuming accidental eccentricity the wall considered for design had the following tributary base shear and moment:

V = 98 kN,M = 990 kNm

The design of the metallic connectors was performed in accordance to NZS 3603 in combination to the connectors' characteristic capacity provided by the manufacturer (see Rothoblaas – Wood Connectors and timber plates).

The CLT panel design was performed in accordance to NZS 3603 and assuming XLam NZ Ltd's products (see XLam NZ Ltd – Cross Laminated Timber Design Guide).

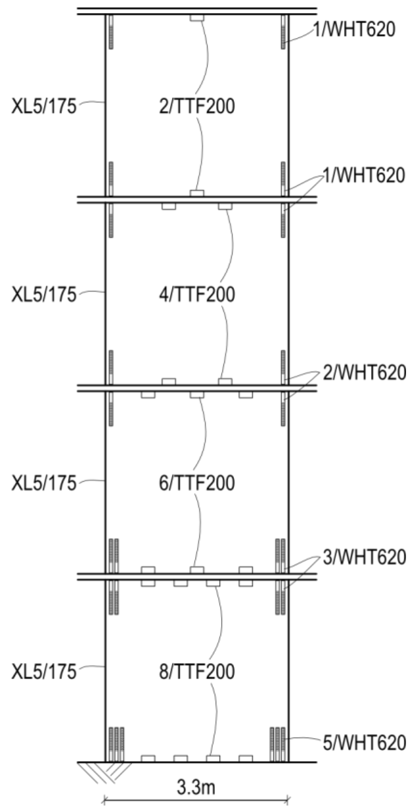


Figure 20. Building A. Sketch of the structural solution.

A.1.2 Nonlinear model

An equivalent truss approach was used to model the CLT walls as shown in Figure 21 and calibrated against experimental results as reported in "Rothoblaas – Seismic-REV – Experimental campaign on Rothoblass products".

The equivalent truss consists of diagonal link elements simulating a single storey panel elastic stiffness (bending and shear). The bracket connections are simulated with link elements limiting the relative in-plane displacement of the panels across different levels. A pinching hysteretic rule was used for these elements. To simulate the rocking mechanism occurring as the panel is subjected to horizontal action a nonlinear link was used and different hysteretic relationships were assigned to the element in the tensile and compressive direction. In compression the hold-down links have an elastic behaviour with high stiffness at ground floor simulating the hard contact against the foundation and with the floor's perpendicular to the grain stiffness at higher levels calibrated according to Blass et al. (2004). As the hold-down connection is loaded in tension, a pinching hysteretic rule was used and calibrated against available experimental results.

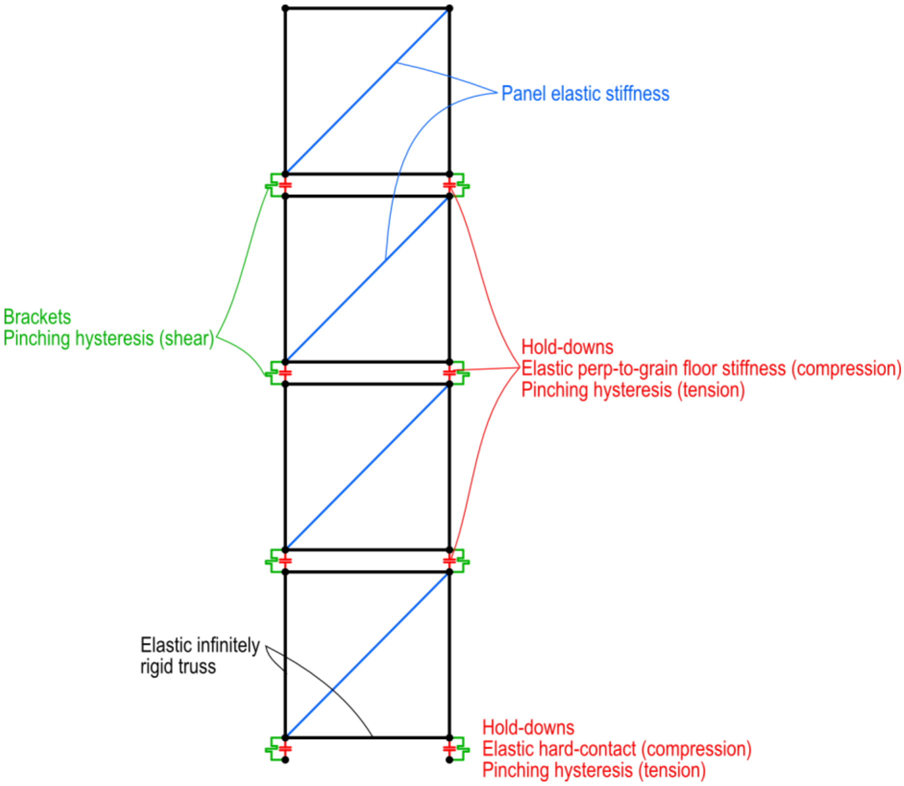


Figure 21. Building A. Sketch of the nonlinear model.

For material properties used in this model refer to manufacturer's literature.

Modal analysis and damping model

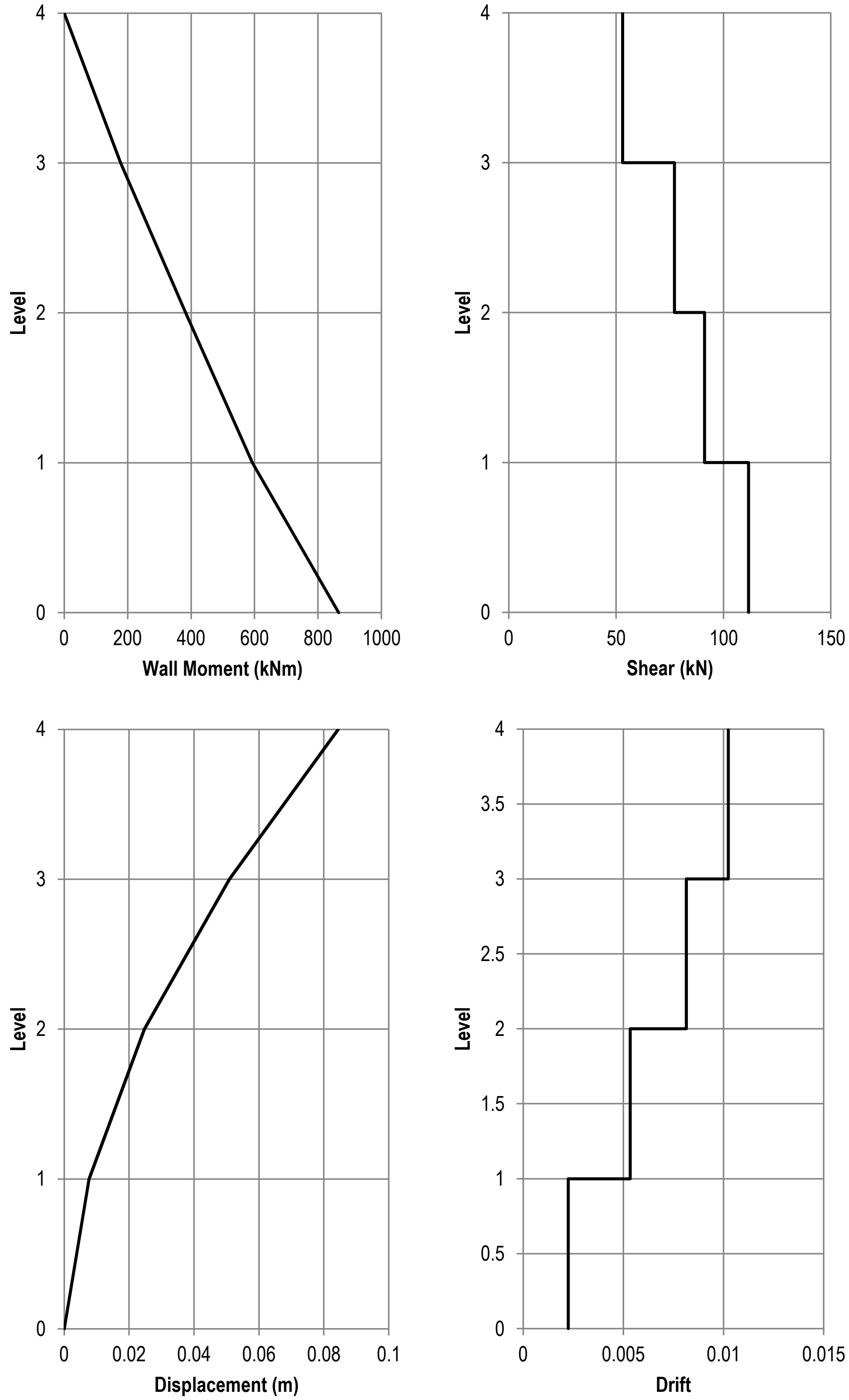
The results of the modal analysis of the Building B nonlinear model are summarized in Table 13.

Mass and tangent stiffness proportional Rayleigh damping model was used in the analyses, and for this building a critical damping ratio of 5% was assigned to the first and second modes of vibration. The resulting modal damping ratios are also reported in Table 13.

Table 13. Building A. Modal analysis results.

No.	T	Part. Mass		Cum. Part. Mass		Damping ratio
	(s)	(t)	(%)	(t)	(%)	(%)
1	0.32	21	77.6	21	77.6	5.0
2	0.12	5	17.8	26	95.4	5.0
3	0.07	1	3.7	27	99.2	6.7
4	0.06	0	0.8	27	100.0	8.1

A.1.3 <u>Nonlinear Time-History Analysis envelopes</u>



A.2 Building B

A.2.1 Lateral load design

Building B is a three-storey concrete office building located in Christchurch. The plan view and elevation of the building are shown in Figure 22.

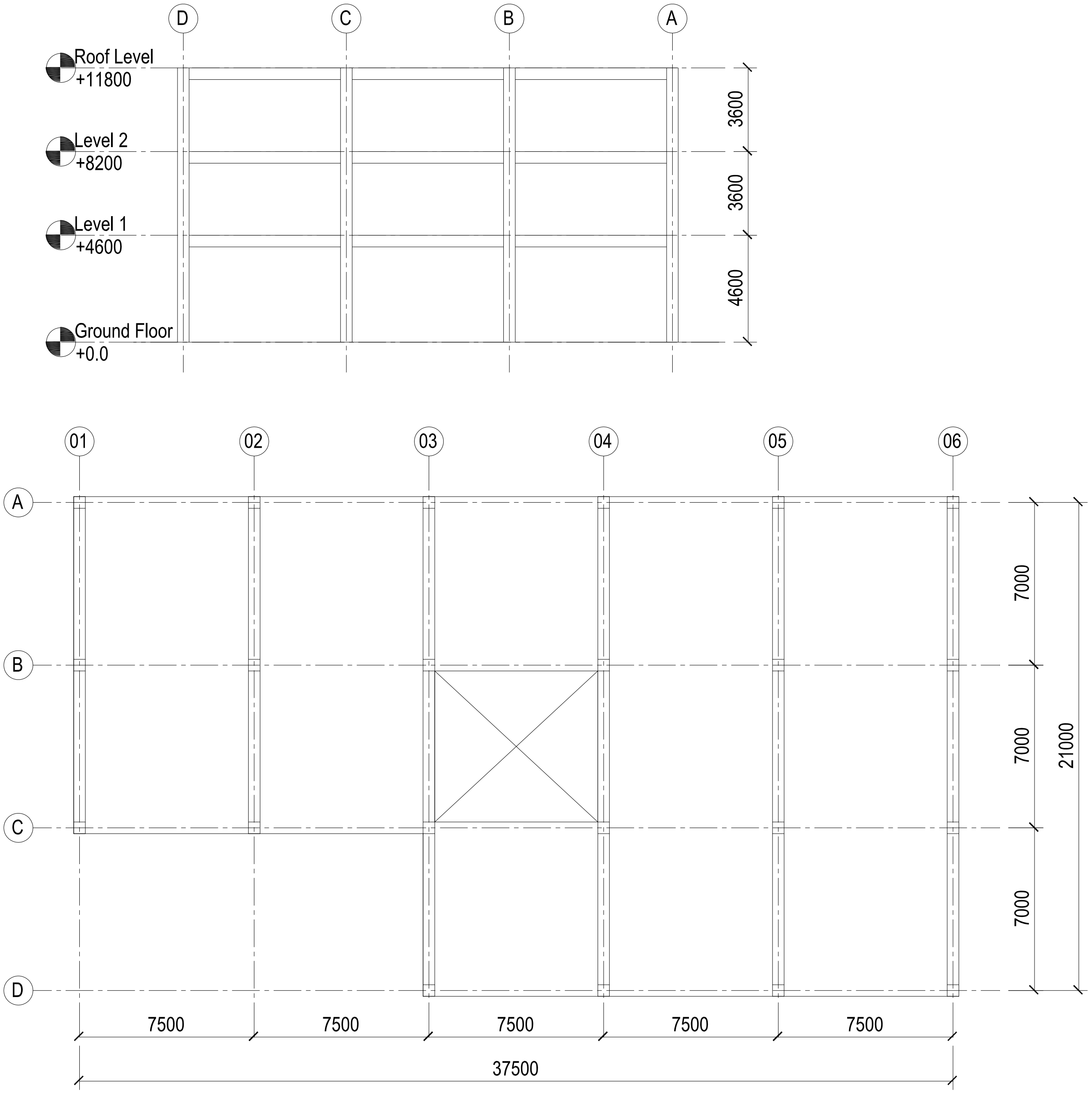


Figure 22. Building B. Plan view and elevation.

Earthquake resistance for this case study building is provided by reinforced concrete frames in the transverse direction (for design and modelling purposes only this direction was considered). The gravity loads are carried by a hollow core flooring systems and the seismic frames.

A summary of the lateral load design parameters is provided in the list below:

- Soil type D
- Hazard factor, Z = 0.30
- Return period factor, R = 1.0

- Near fault factor, N(T,D) = 1.0
- Period of vibration, T_1 = 0.60s
- Structural ductility factor, μ = 2.9
- Structural performance factor, S_p = 0.7
- Inelastic spectrum scaling factor, k_μ = 2.63

The results of the equivalent static method in accordance to NZS 1170.5 are reported in Table 14.

Table 14. Building B. Equivalent static method results.

Level	H_i *(m)*	h_i *(m)*	W_i *(kN)*	W_ih_i *(m·kN)*	F_i/V *(-)*	V_i/V *(-)*	F_i *(kN)*	V_i *(kN)*	M_i *(kNm)*
1	4.6	4.6	4623	21266	0.180	0.180	548	3045	28516
2	3.6	8.2	4563	37417	0.316	0.496	963	2497	14511
3	3.6	11.8	4246	50103	0.504	1.000	1534	1534	5521
		Σ	13432	108785	1.000		3045		

For the purpose of the structural detailing and the numerical analysis the concrete frame on gridline 05 was considered for design and analysis. In accordance to the equivalent static method and assuming accidental eccentricity the wall considered for design has the following tributary base shear and moment:

V = 664 kN, M = 6216 kNm

The concrete frame design was performed in accordance with NZS 3603. Figure 23 shows a sketch of the structural concrete frame reporting the design concrete reinforcement.

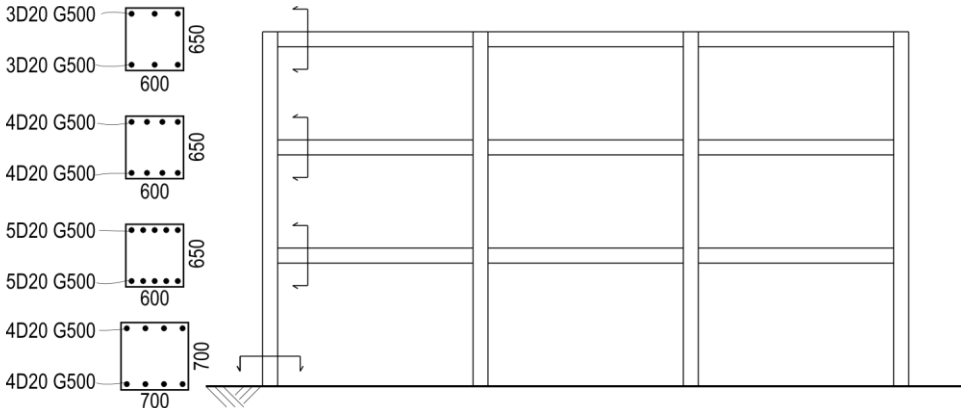


Figure 23. Building B. Sketch of the structural solution.

A.2.2 Nonlinear model

The nonlinear model of Building B is shown in Figure 24. The nonlinear reinforced concrete elements were modelled using fibre elements as outlined below. While these nonlinear fibre elements simulated the nonlinear behaviour of the frame in the plastic hinge areas, elastic elements were used to simulate the capacity protected elements (i.e. columns at higher storeys). Rigid offsets were provided at the beam column joints.

The material properties used in the design and modelling of building B are summarized in Table 15.

Table 15. Building B. Material properties.

Concrete		Reinforcing steel	
Modulus of Elasticity, E_c (GPa)	25	Modulus of elasticity, E_s (GPa)	200
Compressive strength, f'_c (MPa)	30	Yield strength, f_y (MPa)	500
Strain at maximum strength, ε'_c	0.002	Post-yielding ratio, r	0.008

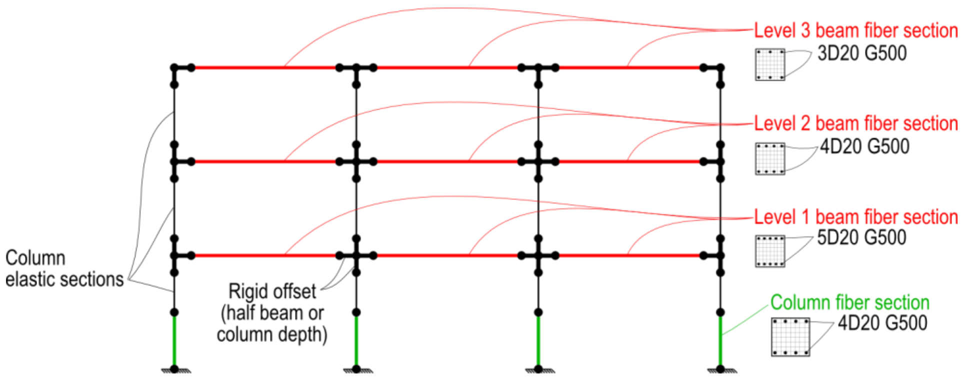


Figure 24. Building B. Sketch of the nonlinear model.

Modal analysis and damping model

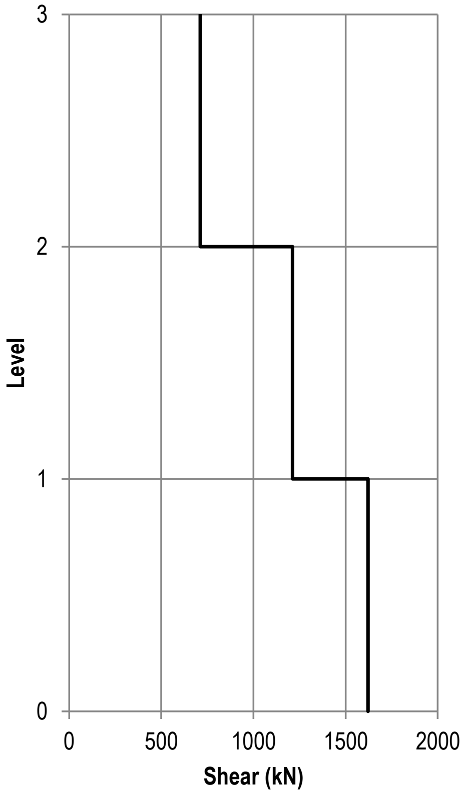
The results of the modal analysis of the Building B nonlinear model are summarized in Table 16.

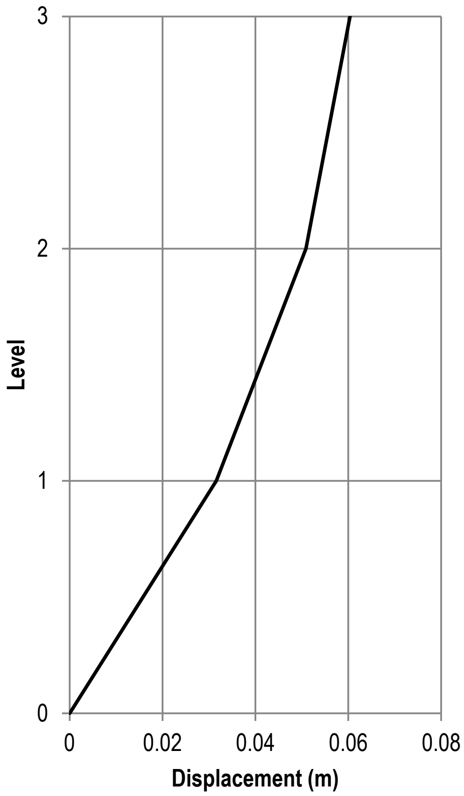
A mass and tangent stiffness proportional Rayleigh damping model was used for the analyses, and for this building a critical damping ratio of 5% was assigned to the first and third modes of vibration. The resulting modal damping ratios are also reported in Table 16.

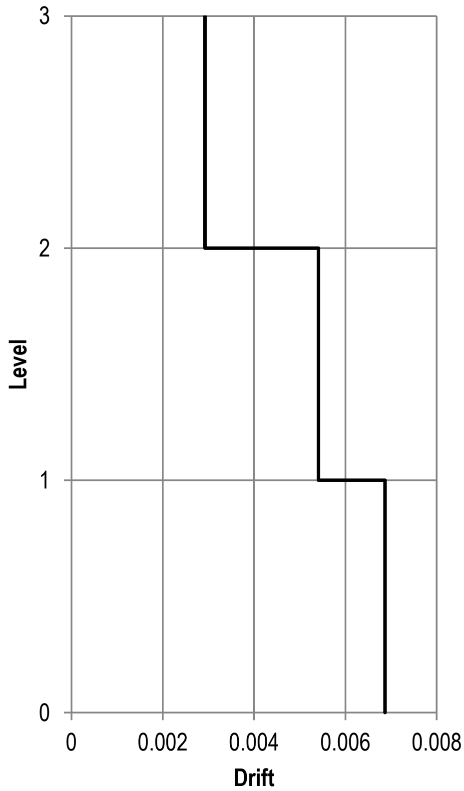
Table 16. Building B. Modal analysis results.

No.	T	Part. Mass		Cum. Part. Mass		Damping ratio
	(s)	(t)	(%)	(t)	(%)	(%)
1	0.47	258	86.3	258	86.3	5.0
2	0.13	35	11.7	293	98.0	3.2
3	0.06	6	2.0	299	100.0	5.0

A.2.3 Nonlinear Time-History Analysis results







A.3 Building C

A.3.1 Lateral load design

Building C is a residential four-storey CLT building located in Wellington. The lateral and vertical load resisting systems are similar to Building A (refer to Section A.1). The plan view and elevation of the building are shown in Figure 25.

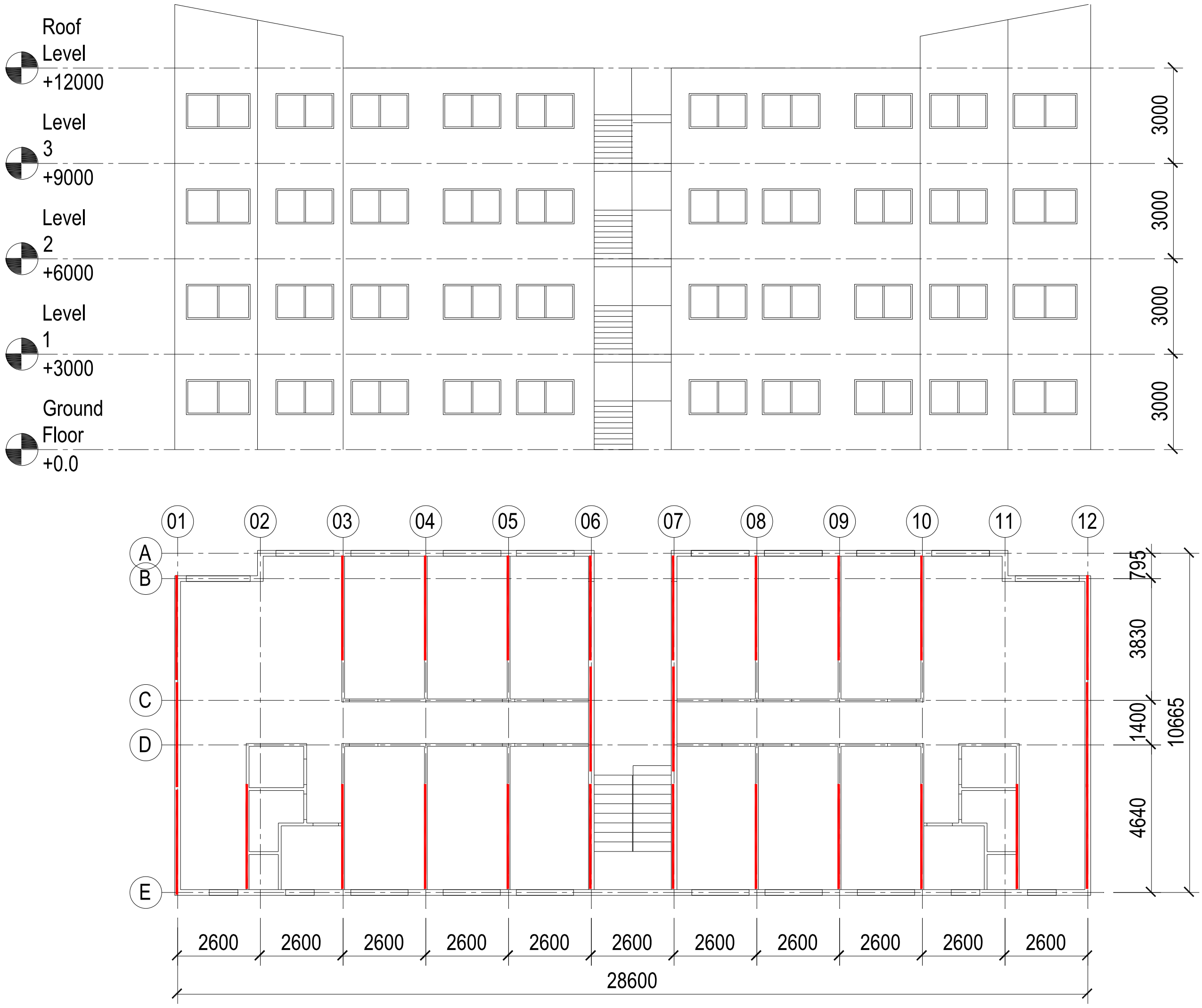


Figure 25. Building C. Plan view and elevation.

The shear walls considered in the lateral load design of the building are highlighted by thick red lines in Figure 25 and consist of 3.3 m CLT wall panels.

A summary of the lateral load design parameters is provided in the list below:

- Soil type C
- Hazard factor, $Z = 0.40$
- Return period factor, $R = 1.0$
- Near fault factor, $N(T,D) = 1.0$
- Period of vibration, $T_1 = 0.50s$
- Structural ductility factor, $\mu = 2.0$
- Structural performance factor, $S_p = 0.7$
- Inelastic spectrum scaling factor, $k_\mu = 1.71$

The results of the equivalent static method in accordance to NZS 1170.5 are reported in Table 17.

Table 17. Building C. Equivalent static method results.

Level	H_i *(m)*	h_i *(m)*	W_i *(kN)*	W_ih_i *(m·kN)*	F_i/V *(-)*	V_i/V *(-)*	F_i *(kN)*	V_i *(kN)*	M_i *(kNm)*
1	3	3	904	2712	0.118	1.000	116	982	8676
2	3	6	764	4584	0.200	0.882	197	866	5730
3	3	9	764	6876	0.300	0.681	295	669	3132
4	3	12	575	6900	0.381	0.381	374	374	1123
		Σ	3007	21072	1.000		982		

For the purpose of this numerical analysis one of the walls on gridline 04 is considered for design and analysis. In accordance to the equivalent static method and assuming accidental eccentricity the tributary shear and moment for the wall considered are:

V = 46 kN, M = 408 kNm

The design of the metallic connectors was performed in accordance to NZS 3603 and considering connectors' characteristic capacity provided by the manufacturer (see Rothoblaas – Wood Connectors and timber plates).

The CLT panel design was performed in accordance to NZS 3603 and assuming XLam NZ Ltd's products (see XLam NZ Ltd – Cross Laminated Timber Design Guide).

The structural design of the CLT shear wall was performed in accordance to NZS 3603 and the resulting mechanical connectors configuration in shown in Figure 26.

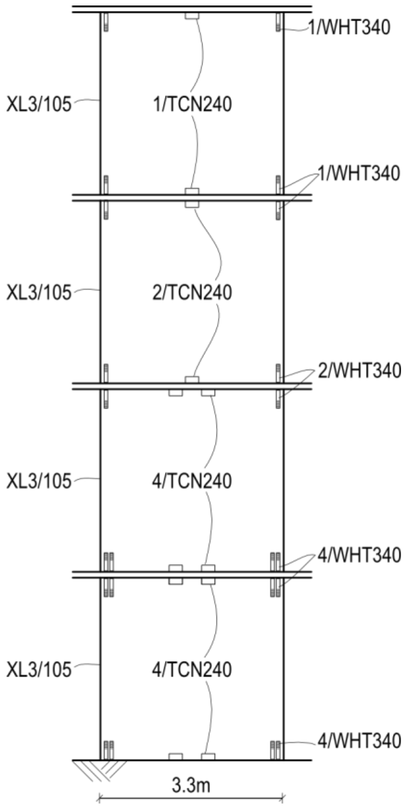


Figure 26. Building C. Sketch of the structural solution.

A.3.2 Nonlinear model

The nonlinear model of Building C was assembled and calibrated similarly to the nonlinear model of Building A. Section A.1.2 provides an overview of the nonlinear model development and calibration

Modal analysis and damping model

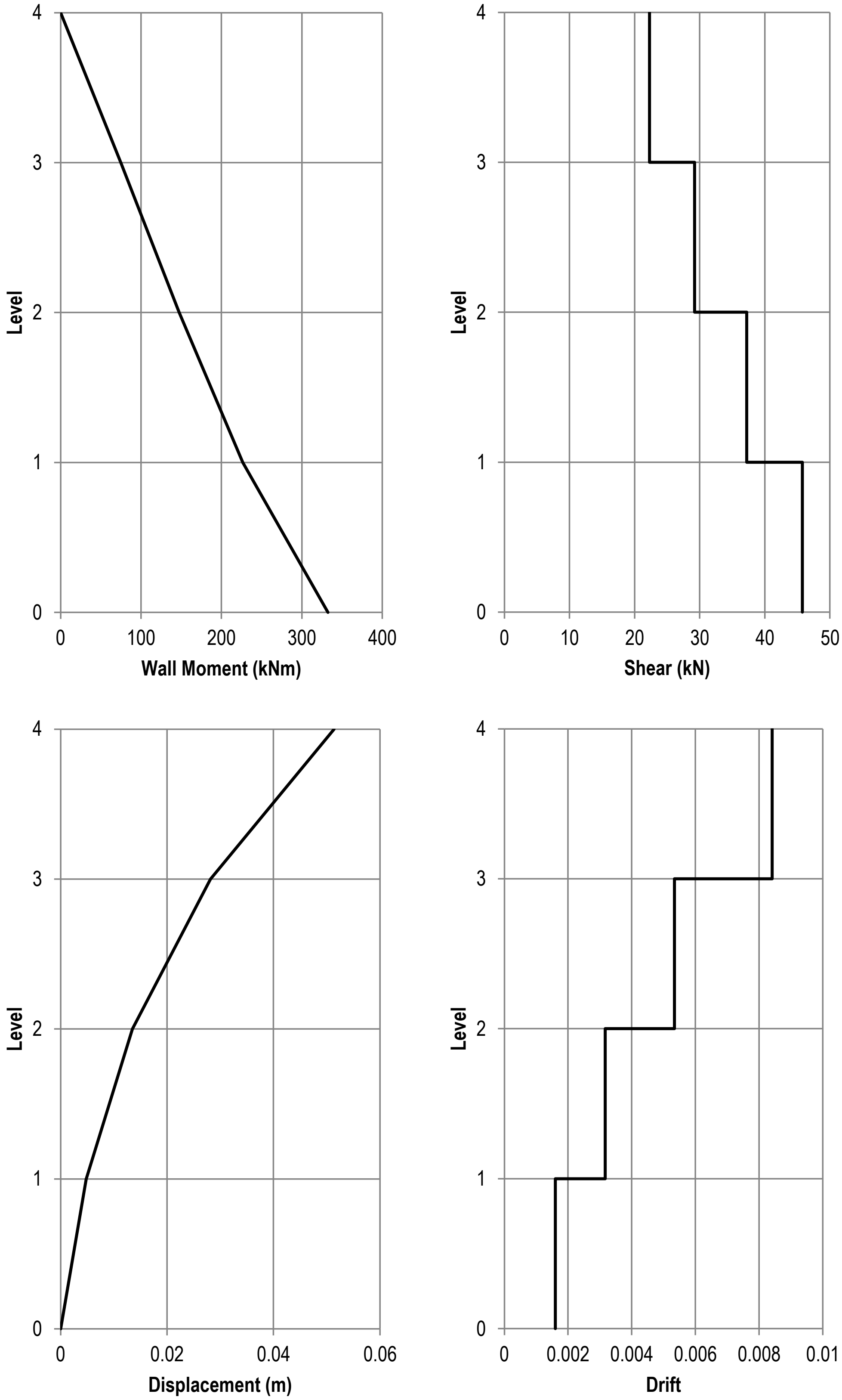
The results of the modal analysis of the Building C nonlinear model are summarized in Table 18.

Mass and tangent proportional Rayleigh damping model was used in the analyses, and for this building a critical damping ratio of 5% was assigned to the first and second modes of vibration. The resulting modal damping ratios are also reported in Table 18.

Table 18. Building C. Modal analysis results.

No.	T	Part. Mass		Cum. Part. Mass		Damping ratio
	(s)	(t)	(%)	(t)	(%)	(%)
1	0.30	11	77.5	11	77.5	5.0
2	0.12	2	16.1	13	93.6	5.0
3	0.08	1	4.6	14	98.2	6.4
4	0.06	0	1.8	14	100.0	8.0

A.3.3 Nonlinear Time-History Analysis results



A.4 Building D

A.4.1 Lateral load design

Building D is a twelve-story office building with a residential penthouse at the top level located in Wellington. The plan view and the elevation of the structure are shown in Figure 27.

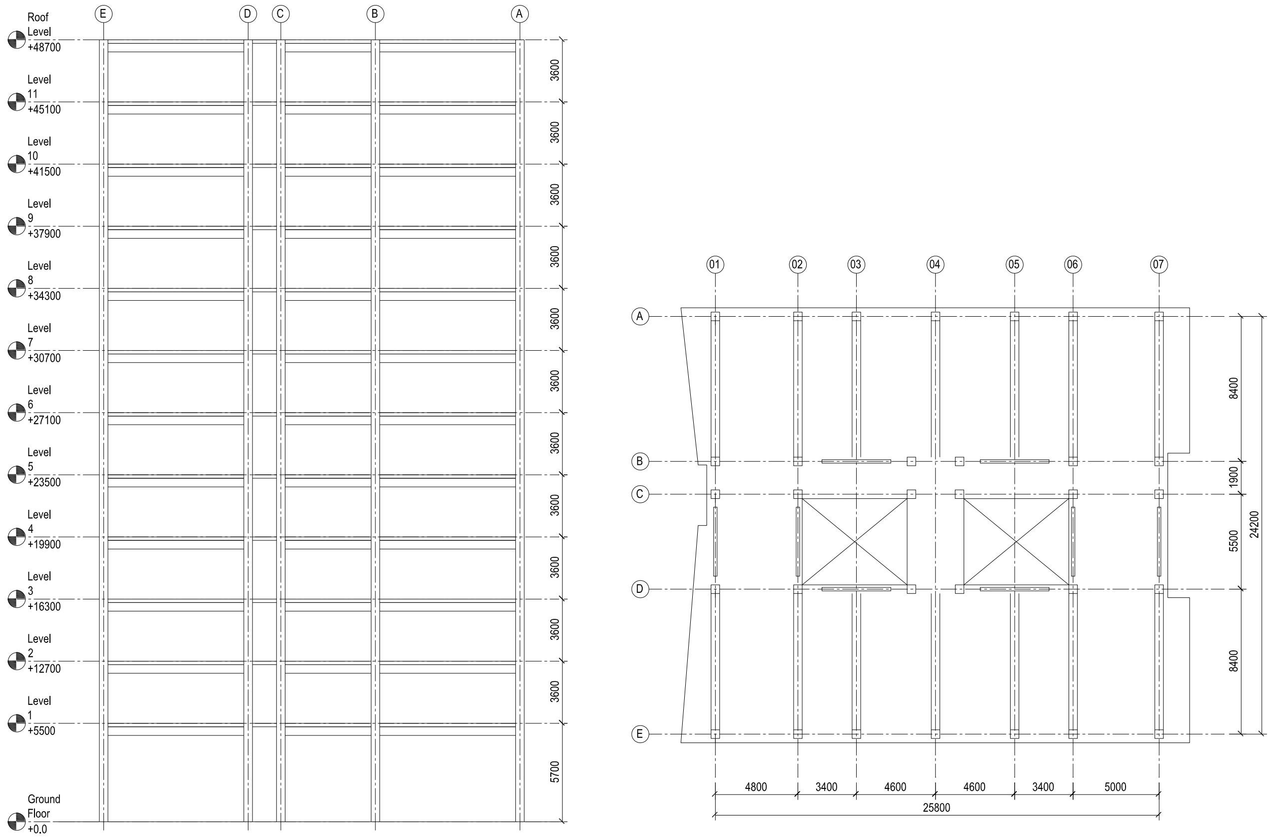


Figure 27. Building D. Plan view and elevation.

The building's lateral load resisting system in the North-South direction (the direction considered for design and analysis) consists of a combination of reinforced concrete walls and a reinforced concrete frames on gridlines 01, 02, 06 and 07 (refer to Figure 27). Gravity loads are carried by a rib and infill flooring system supported by reinforced concrete frames.

A summary of the lateral load design parameters is provided in the list below:

- Soil type C
- Hazard factor, $Z = 0.40$
- Return period factor, $R = 1.0$
- Near fault factor, $N(T,D) = 1.0$
- Period of vibration, $T_1 = 1.24s$
- Structural ductility factor, $\mu = 3.0$
- Structural performance factor, $S_p = 0.7$
- Inelastic spectrum scaling factor, $k_\mu = 3.0$

The results of the equivalent static method in accordance to NZS 1170.5 are reported in Table 19.

Table 19. Building D. Equivalent static method results.

Level	H_i (m)	h_i (m)	W_i (kN)	W_ih_i (m·kN)	F_i/V (-)	V_i/V (-)	F_i (kN)	V_i (kN)	M_i (kNm)
1	5.5	5.5	12468	68574	0.026	1.000	248	9459	300752
2	3.6	9.1	8152	74183	0.028	0.974	268	9212	248726
3	3.6	12.7	8152	103530	0.040	0.945	374	8943	215565
4	3.6	16.3	8152	132878	0.051	0.906	480	8569	183368
5	3.6	19.9	8152	162225	0.062	0.855	586	8089	152518
6	3.6	23.5	8152	191572	0.073	0.793	692	7503	123397
7	3.6	27.1	8152	220919	0.084	0.720	798	6811	96385
8	3.6	30.7	8152	250266	0.096	0.636	904	6013	71865
9	3.6	34.3	8152	279614	0.107	0.540	1010	5109	50218
10	3.6	37.9	8152	308961	0.118	0.433	1116	4099	31827
11	3.6	41.5	8152	338308	0.129	0.315	1222	2982	17072
12	3.6	45.1	6157	277681	0.186	0.186	1760	1760	6336
		Σ	100145	2408711	1.000		9459		

For design and modelling purposes the hybrid wall-frame system located on gridline 06 was considered. According to the equivalent static method and accounting for accidental eccentricity in accordance to NZS 1170.5 the following tributary base shear and moment result:

V = 3,103 kN, M = 98,647 kNm

The reinforcement of the hybrid wall-frame concrete structure was designed in accordance to NZS 3101 and the resulting flexural reinforcement of the structural elements is summarized in Figure 28.

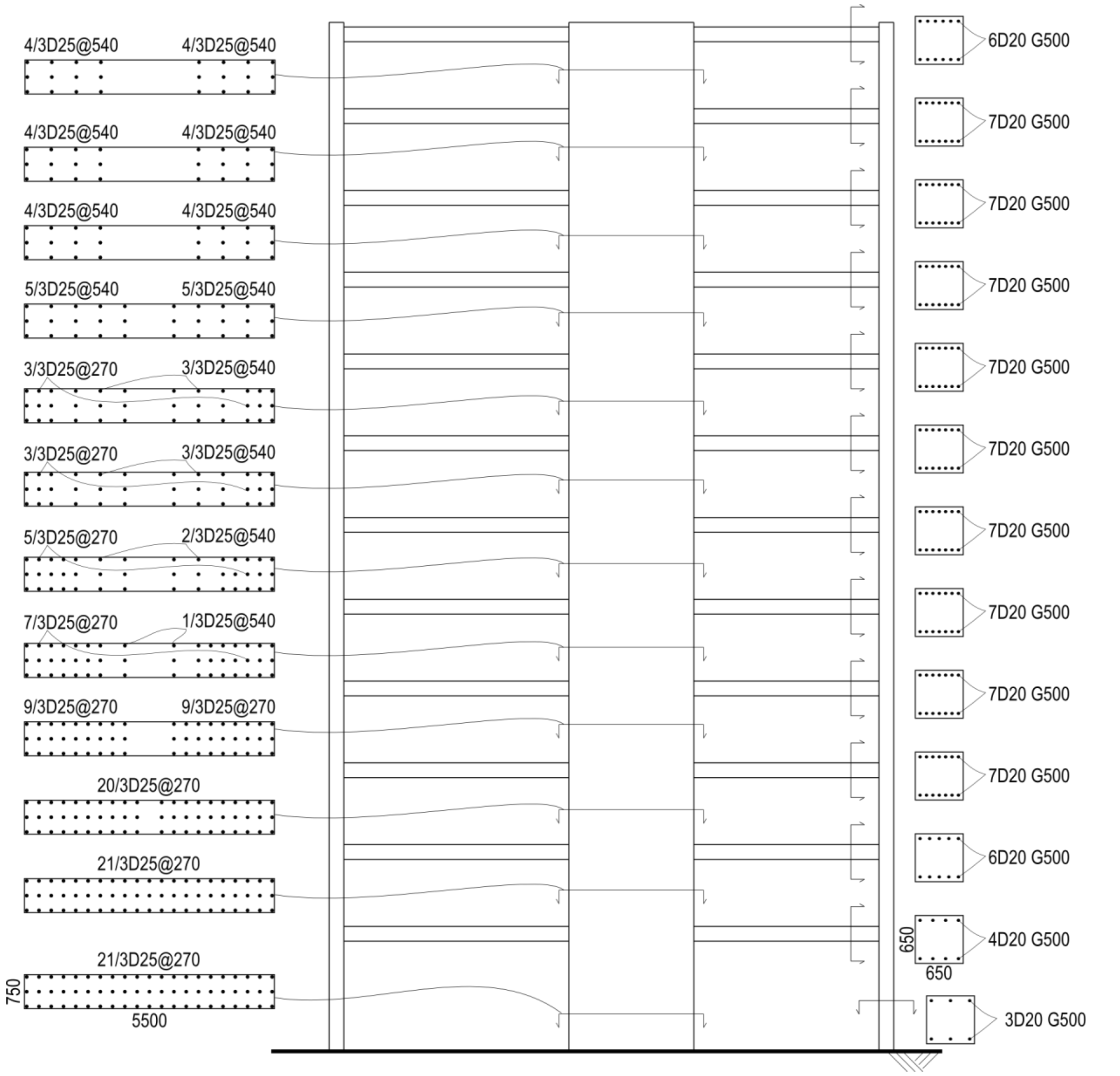


Figure 28. Building D. Sketch of the structural solution.

A.4.2 Nonlinear model

The nonlinear model of Building D is shown in Figure 28. The hybrid system was modelled using nonlinear fibre sections which were used for the beams and the reinforced concrete wall, and the potential plastic hinge areas of the columns. The capacity protected areas of the columns were modelled as linear elastic elements. Rigid offsets were provided at the beam column joints as well as at the connection between the wall and the concrete beams.

The material properties used in the design and modelling of building B are summarized in Table 20.

Table 20. Building D. Material properties.

Concrete		Reinforcing steel	
Modulus of Elasticity, E_c (GPa)	25	Modulus of elasticity, E_s (GPa)	200
Compressive strength, f'_c (MPa)	30	Yield strength, f_y (MPa)	500
Strain at maximum strength, ε'_c	0.002	Post-yielding ratio, r	0.008

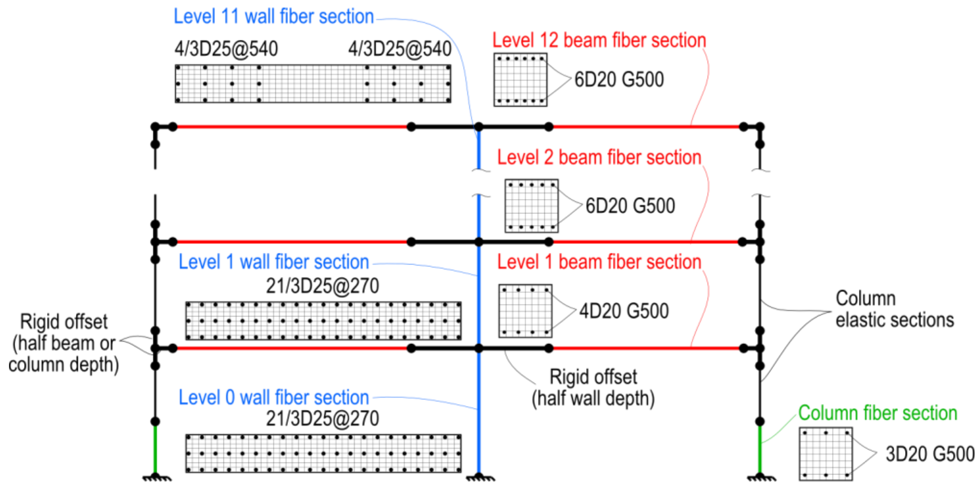


Figure 11. Building D. Sketch of the nonlinear model.

Modal analysis and damping model

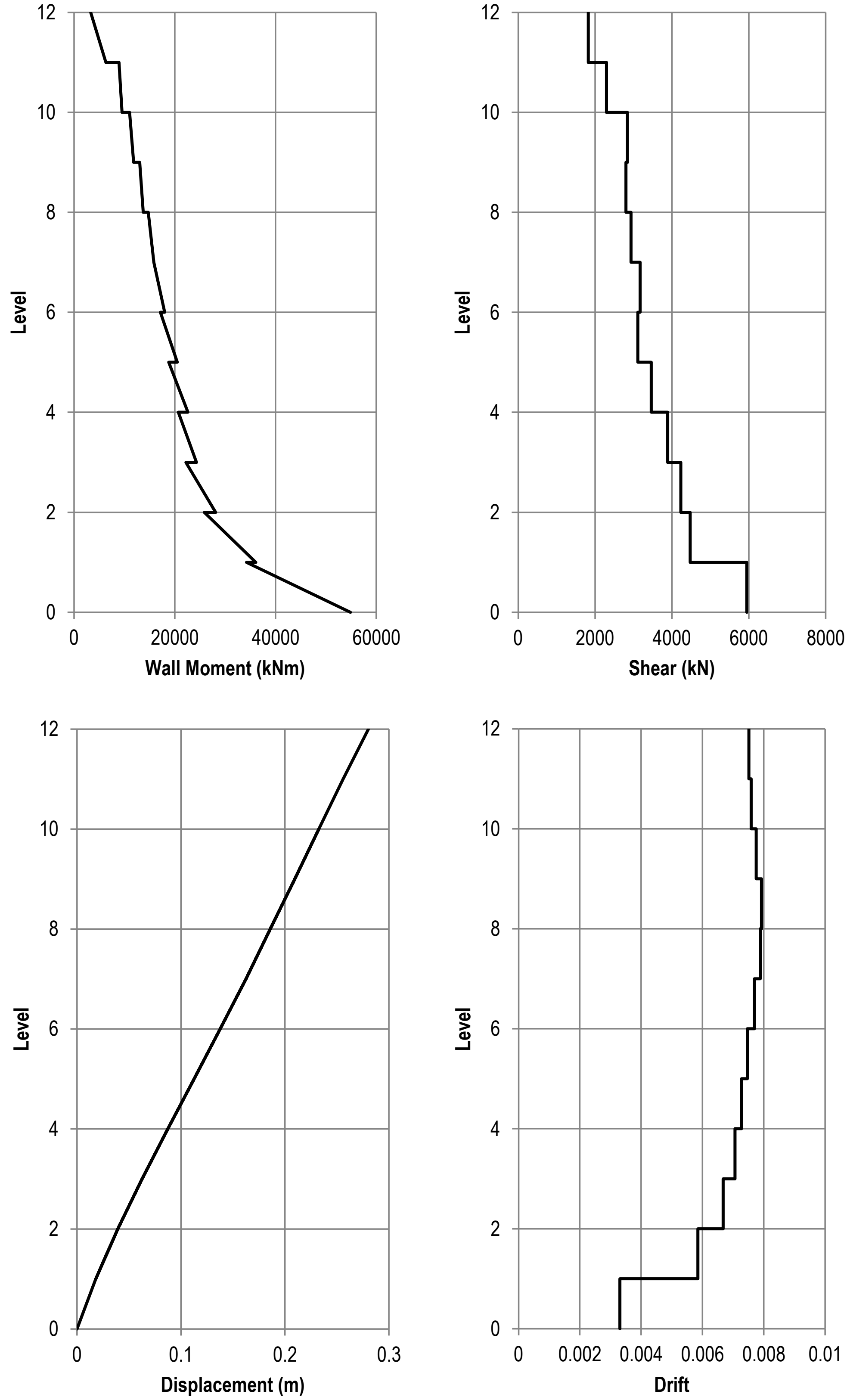
The results of the modal analysis performed on building D is reported in Table 21, which also summarizes the Rayleigh modal damping ratios.

A damping ratio of 5% was set for the first and third modes of vibration which results in the modal damping ratios below.

Table 21. Building D. Modal analysis results.

No.	T	Part. Mass		Cum. Part. Mass		Damping ratio
	(s)	(t)	(%)	(t)	(%)	(%)
1	1.446	2191	65.5	2191	65.5	5.0
2	0.289	605	18.1	2795	83.6	2.7
3	0.110	243	7.3	3038	90.8	5.0
4	0.057	138	4.1	3176	95.0	9.1
5	0.035	83	2.5	3259	97.4	14.5
6	0.024	46	1.4	3305	98.8	21.3
7	0.017	22	0.7	3327	99.5	29.5
8	0.013	10	0.3	3337	99.8	39.3
9	0.010	5	0.1	3342	99.9	50.2
10	0.008	2	0.1	3344	100.0	61.6
11	0.007	1	0.0	3345	100.0	72.1
12	0.006	0	0.0	3345	100.0	79.8

A.4.3 Nonlinear Time-History Analysis results



A.5 Building E

A.5.1 Lateral load design

Building E is a residential seven-story building located in Auckland. Lateral loads are resisted by two C-shaped reinforced concrete walls in the two directions. Vertical loads are carried by precast hollow core flooring system supported by the reinforced frame structure in the longitudinal direction of the building.

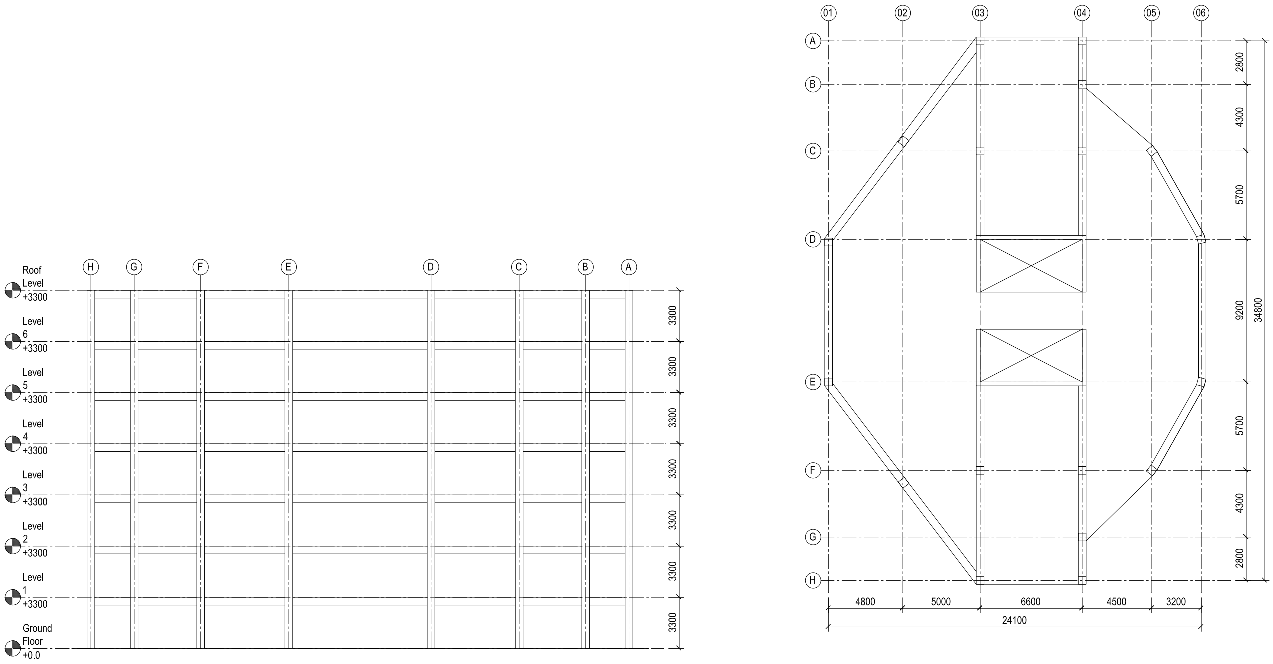


Figure 29. Building E. Plan view and elevation.

A summary of the lateral load design parameters is provided in the list below:

- Soil type B
- Hazard factor, Z = 0.13
- Return period factor, R = 1.0
- Near fault factor, N(T,D) = 1.0
- Period of vibration, T_1 = 0.80s
- Structural ductility factor, μ = 3.0
- Structural performance factor, S_p = 0.7
- Inelastic spectrum scaling factor, k_μ = 3.0

The results of the equivalent static method in accordance to NZS 1170.5 are reported in Table 22.

Table 22. Building E. Equivalent static method results.

Level	H_i *(m)*	h_i *(m)*	W_i *(kN)*	W_ih_i *(m·kN)*	F_i/V *(-)*	V_i/V *(-)*	F_i *(kN)*	V_i *(kN)*	M_i *(kNm)*
1	3.3	3.3	8671	28614	0.038	1.000	70	1828	30646
2	3.3	6.6	7656	50530	0.068	0.962	124	1758	24612
3	3.3	9.9	7656	75794	0.101	0.894	185	1635	18810
4	3.3	13.2	7656	101059	0.135	0.793	247	1449	13416
5	3.3	16.5	7656	126324	0.169	0.657	309	1202	8633
6	3.3	19.8	7656	151589	0.203	0.488	371	893	4667
7	3.3	23.1	6641	153407	0.285	0.285	522	522	1722
		Σ	53592	687317	1.000		1828		

For design and modelling purposes the transvers direction (i.e. East-West direction) was considered for Building E and the tributary base shear and moment are:

V = 3103 kN, M = 98647 kNm

The reinforced concrete wall was designed to NZS 3101 and the following flexural reinforcement was evaluated.

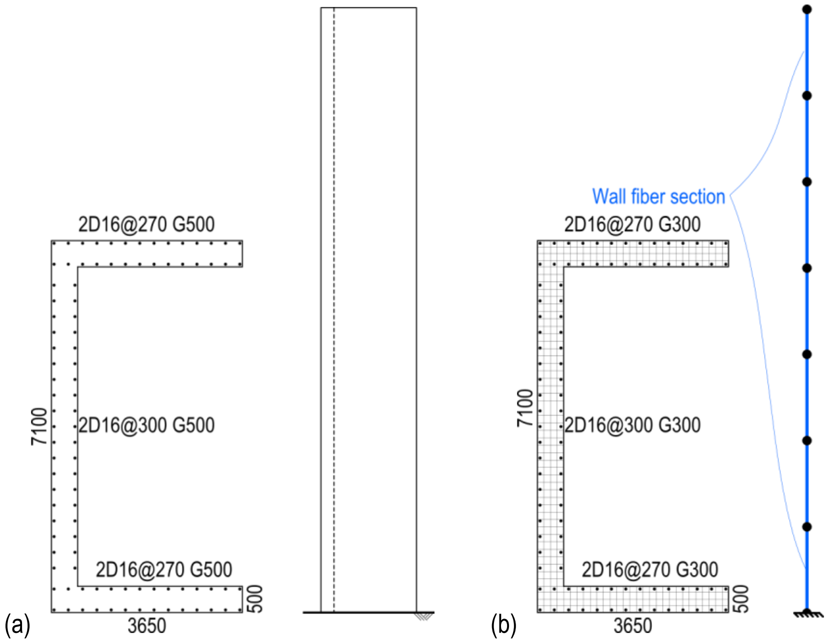


Figure 30. Building E. Sketches of (a) the structural solution and (b) the nonlinear model.

A.5.2 Nonlinear model

Nonlinear fibre elements were used to simulate the inelastic behaviour of the reinforced concrete walls as shown in Figure 30b.

The material properties used in the design and modelling of building E are summarized in Table 23.

Table 23. Building E. Material properties.

Concrete		Reinforcing steel	
Modulus of Elasticity, E_c (GPa)	25	Modulus of elasticity, E_s (GPa)	200
Compressive strength, f'_c (MPa)	30	Yield strength, f_y (MPa)	300
Strain at maximum strength, ε'_c	0.002	Post-yielding ratio, r	0.008

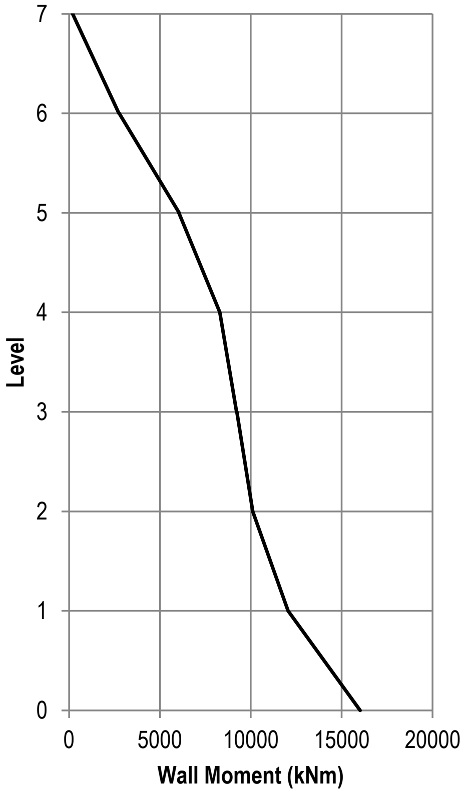
Modal analysis and damping model

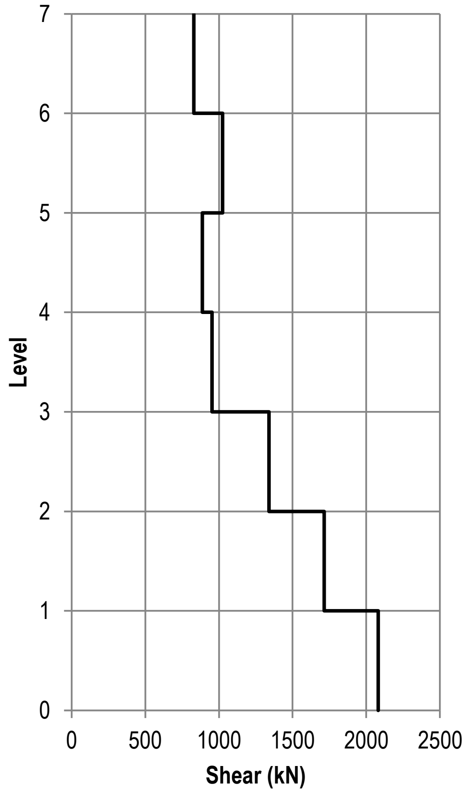
The modal analysis results for Building E are shown in Table 24. Mass and tangent stiffness proportional Rayleigh damping ratios were set to 5% at first and third mode, and the resulting modal damping ratios are shown in Table 24.

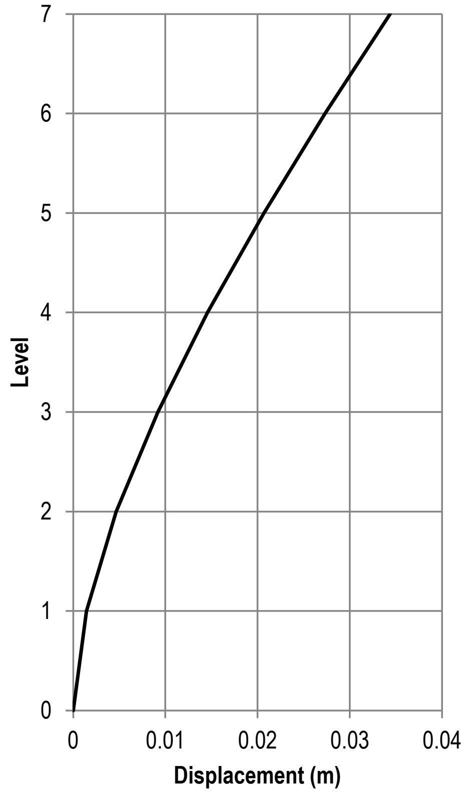
Table 24. Building E. Modal analysis results.

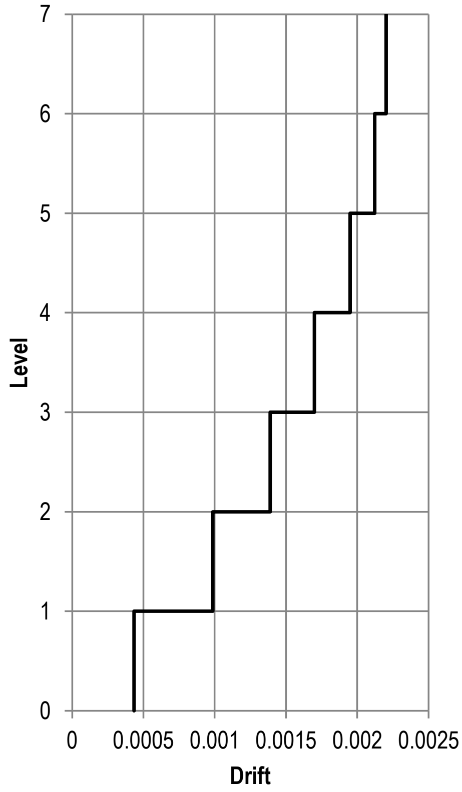
No.	T	Part. Mass		Cum. Part. Mass		Damping ratio
	(s)	(t)	(%)	(t)	(%)	(%)
1	0.32	1767	64.7	1767	64.7	5.0
2	0.05	555	20.3	2322	85.0	2.5
3	0.02	202	7.4	2524	92.4	5.0
4	0.01	108	3.9	2632	96.3	9.3
5	0.01	61	2.2	2693	98.6	15.0
6	0.00	30	1.1	2724	99.7	21.4
7	0.00	9	0.3	2733	100.0	26.9

A.5.3 Nonlinear Time-History Analysis results









A.6 Building F

A.6.1 Lateral load design

Building F is an eight-storey office building located in Christchurch. A structural steel framing with corrugated metal-concrete flooring system carries the gravity loads, and lateral stability is provided by Buckling Restrained Braces (BRBs). The plan view and elevation of the building are shown in Figure 31.

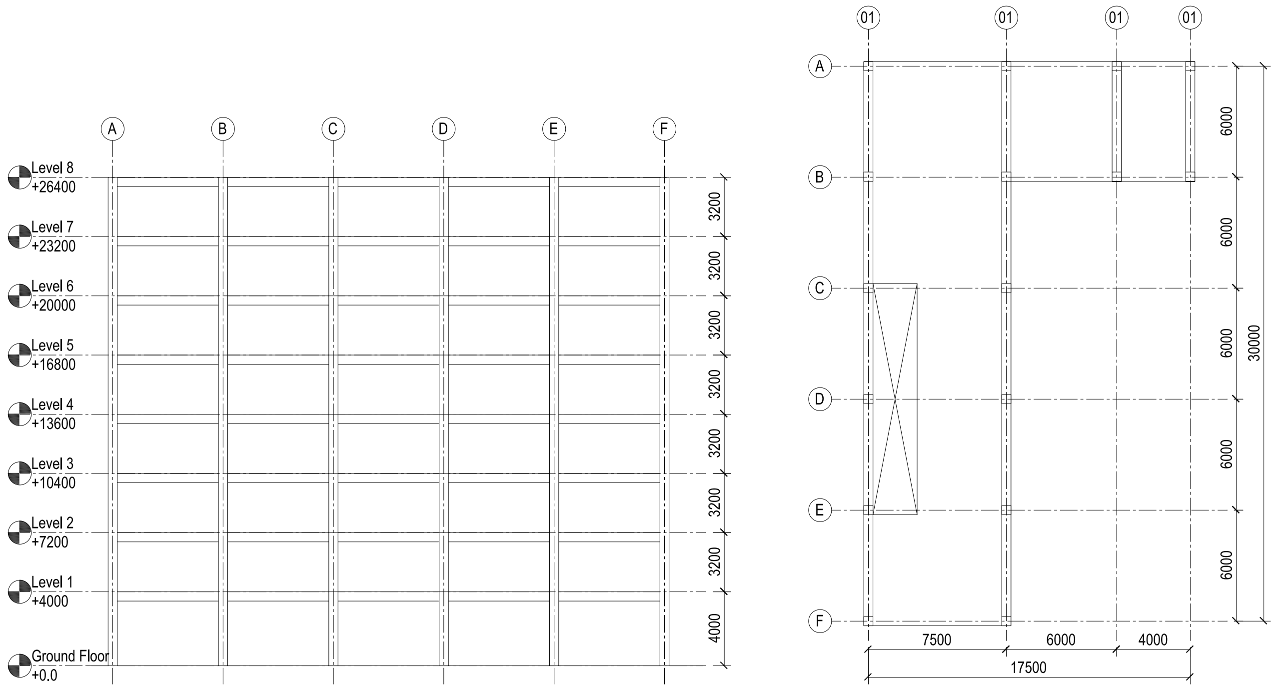


Figure 31. Building F. Plan view and elevation.

A summary of the lateral load design parameters is provided in the list below:

- Soil type D
- Hazard factor, $Z = 0.30$
- Return period factor, $R = 1.0$
- Near fault factor, $N(T,D) = 1.0$
- Period of vibration, $T_1 = 0.87s$
- Structural ductility factor, $\mu = 3.0$
- Structural performance factor, $S_p = 0.7$
- Inelastic spectrum scaling factor, $k_\mu = 3.0$

The results of the equivalent static method in accordance to NZS 1170.5 are reported in Table 25.

Table 25. Building F. Equivalent static method results.

Level	H_i *(m)*	h_i *(m)*	W_i *(kN)*	W_ih_i *(m·kN)*	F_i/V *(-)*	V_i/V *(-)*	F_i *(kN)*	V_i *(kN)*	M_i *(kNm)*
1	4	4	1836	7344	0.036	1.000	69	1899	36055
2	3.2	7.2	1585	11412	0.056	0.964	107	1830	28457
3	3.2	10.4	1585	16484	0.082	0.907	155	1723	22600
4	3.2	13.6	1585	21556	0.107	0.826	203	1568	17086
5	3.2	16.8	1585	26628	0.132	0.719	250	1366	12068
6	3.2	20	1585	31700	0.157	0.587	298	1116	7697
7	3.2	23.2	1585	36772	0.182	0.431	346	818	4128
8	3.2	26.4	1291	34082	0.249	0.249	472	472	1511
		Σ	12637	185978	1.000		1899		

The base shear and moment demands were evaluated with the equivalent static method in accordance to NZS 1170.5 and the tributary base shear and moment for the BRB frame on gridline 01 are.

V = 475 kN, M = 9014 kNm

The steel frame was performed in accordance to NZS 3404 and the resulting structural sections are shown in Figure 32a.

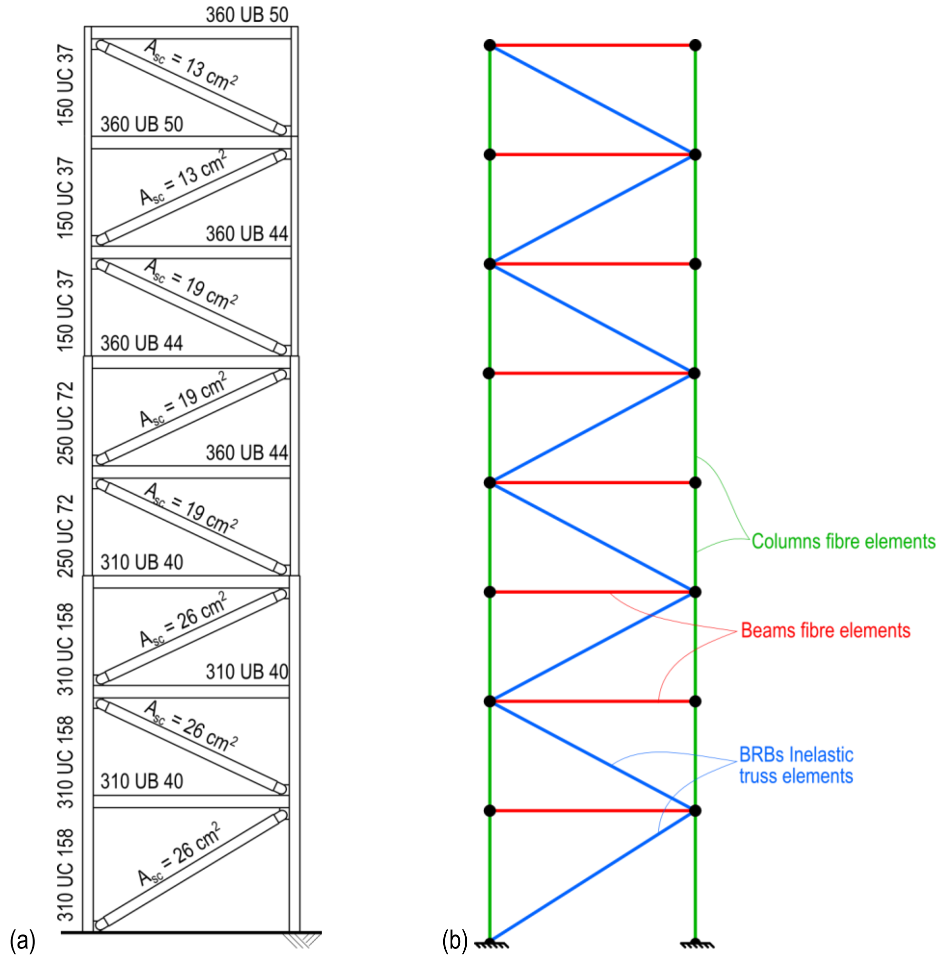


Figure 32. Building F. Sketches of (a) the structural solution and (b) the nonlinear model.

A.6.2 Nonlinear model

A sketch of the nonlinear model of Building F is shown in Figure 32b. All structural elements were modelled as nonlinear. The columns were assumed continuous along the building's height and were modelled using nonlinear fibre elements. The beams were modelled as simply supported nonlinear fibre elements. The BRBs were modelled as inelastic truss elements.

Table 26. Building F. Material properties.

Structural Steel		BRB steel	
Modulus of elasticity, E_s (GPa)	200	Modulus of elasticity, E_s (GPa)	200
Yield strength, f_y (MPa)	300	Yield strength, f_y (MPa)	250
Post-yielding ratio, r	0.008	Post-yielding ratio, r	0.008

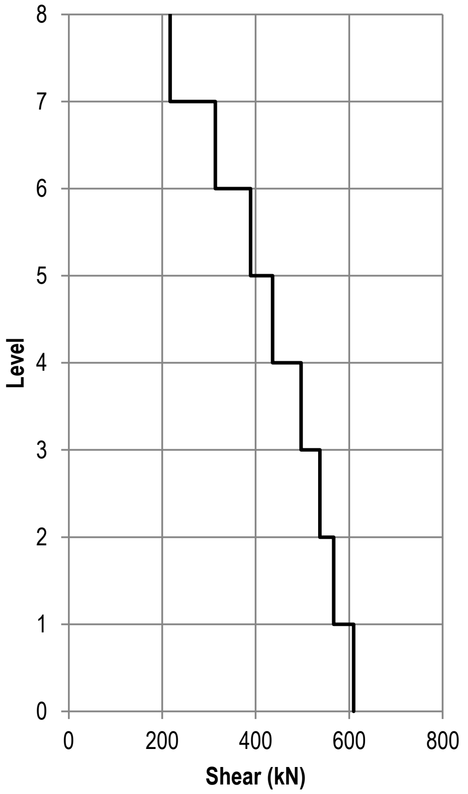
Modal analysis and damping model

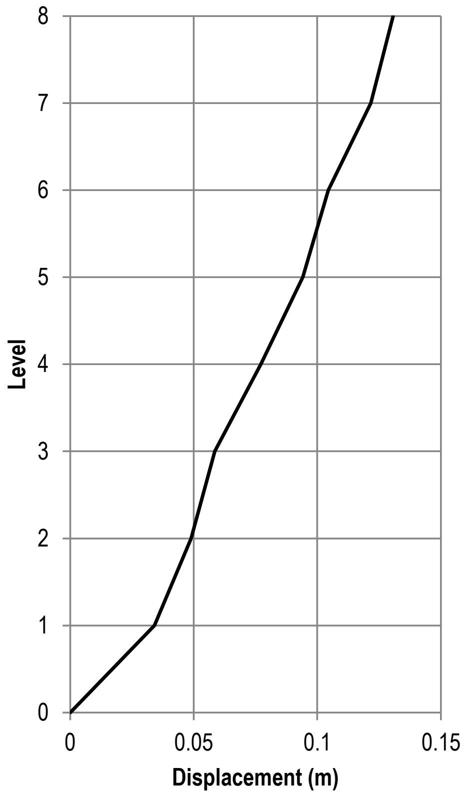
The modal properties of the building are summarized in Table 27. The damping ratios are also reported in the table below.

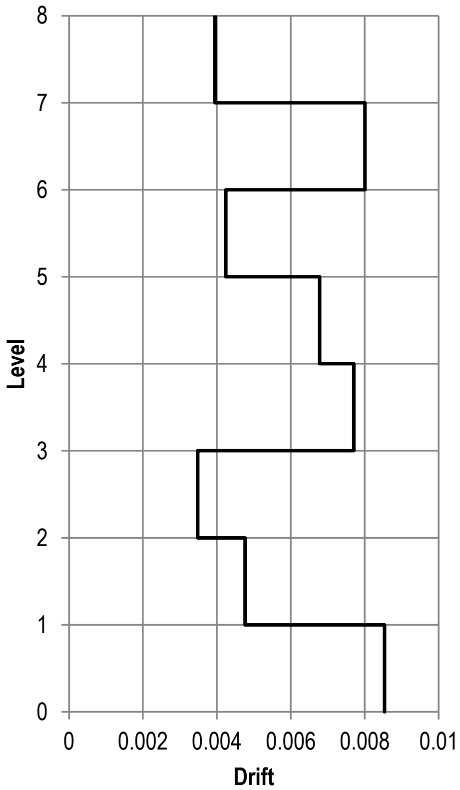
Table 27. Building F. Modal analysis results.

No.	T	Part. Mass		Cum. Part. Mass		Damping ratio
	(s)	(t)	(%)	(t)	(%)	(%)
1	0.98	238	73.9	238	73.9	5.0
2	0.33	59	18.4	297	92.3	5.0
3	0.19	16	4.9	313	97.2	7.2
4	0.14	5	1.5	318	98.7	9.2
5	0.12	2	0.6	320	99.3	10.5
6	0.11	1	0.5	321	99.8	11.7
7	0.10	0	0.1	322	99.9	13.0
8	0.09	0	0.1	322	100.0	14.7

A.6.3 Nonlinear Time-History Analysis results







A.7 Building G

A.7.1 Lateral load design

Building G is a nine-storey office structure located in Auckland. Eccentrically-Braced Frames provide the lateral stability to the structure and a structural steel frame with corrugated metal deck flooring carries the vertical loads. The plan view and elevation of the structure is shown in Figure 33.

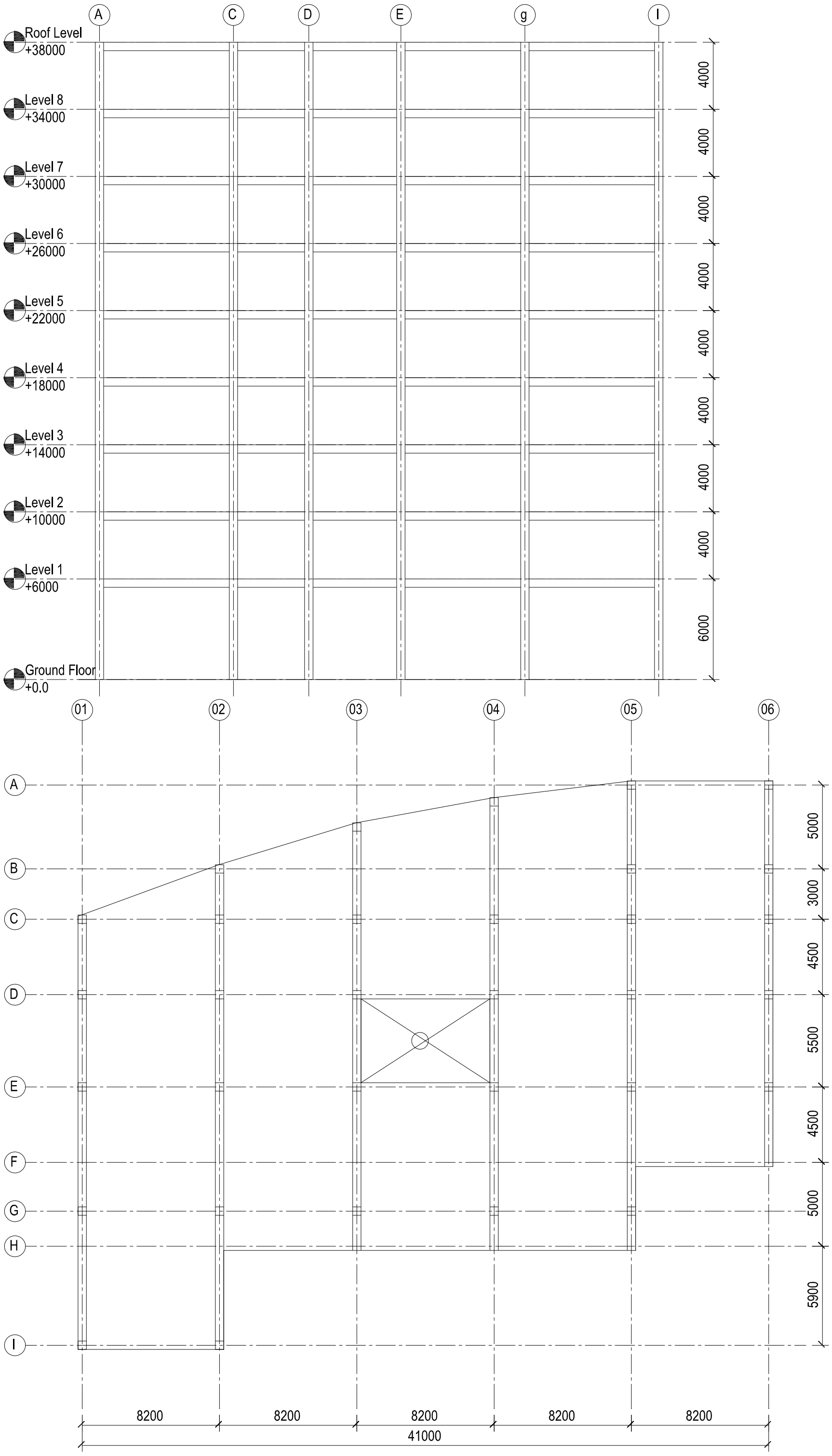


Figure 33. Building A. Plan view and elevation.

A summary of the lateral load design parameters is provided in the list below:

- Soil type B
- Hazard factor, Z = 0.13
- Return period factor, R = 1.0
- Near fault factor, N(T,D) = 1.0
- Period of vibration, T_1 = 1.25s
- Structural ductility factor, μ = 2.0
- Structural performance factor, S_p = 0.7
- Inelastic spectrum scaling factor, k_μ = 2.0

The results of the equivalent static method in accordance to NZS 1170.5 are reported in Table 28.

Table 28. Building G. Equivalent static method results.

Level	H_i *(m)*	h_i *(m)*	W_i *(kN)*	W_ih_i *(m·kN)*	F_i/V *(-)*	V_i/V *(-)*	F_i *(kN)*	V_i *(kN)*	M_i *(kNm)*
1	6	6	5848	35088	0.032	1.000	58	1809	49983
2	4	10	5080	50800	0.046	0.968	84	1751	39127
3	4	14	5080	71120	0.065	0.922	117	1667	32122
4	4	18	5080	91440	0.083	0.857	151	1550	25452
5	4	22	5080	111760	0.102	0.773	185	1399	19252
6	4	26	5080	132080	0.121	0.671	218	1214	13656
7	4	30	5080	152400	0.139	0.551	252	996	8799
8	4	34	5080	172720	0.158	0.411	285	744	4814
9	4	38	5009	190342	0.254	0.254	459	459	1837
		Σ	46417	1007750	1.000		1809		

The frame on Gridline 02 was considered for analysis and it was designed to NZS 1170.5. The resulting shear and moment demands are

V = 520 kN, M = 14370 kNm

The resulting structural design in accordance to NZS3404 is shown in Figure 34a.

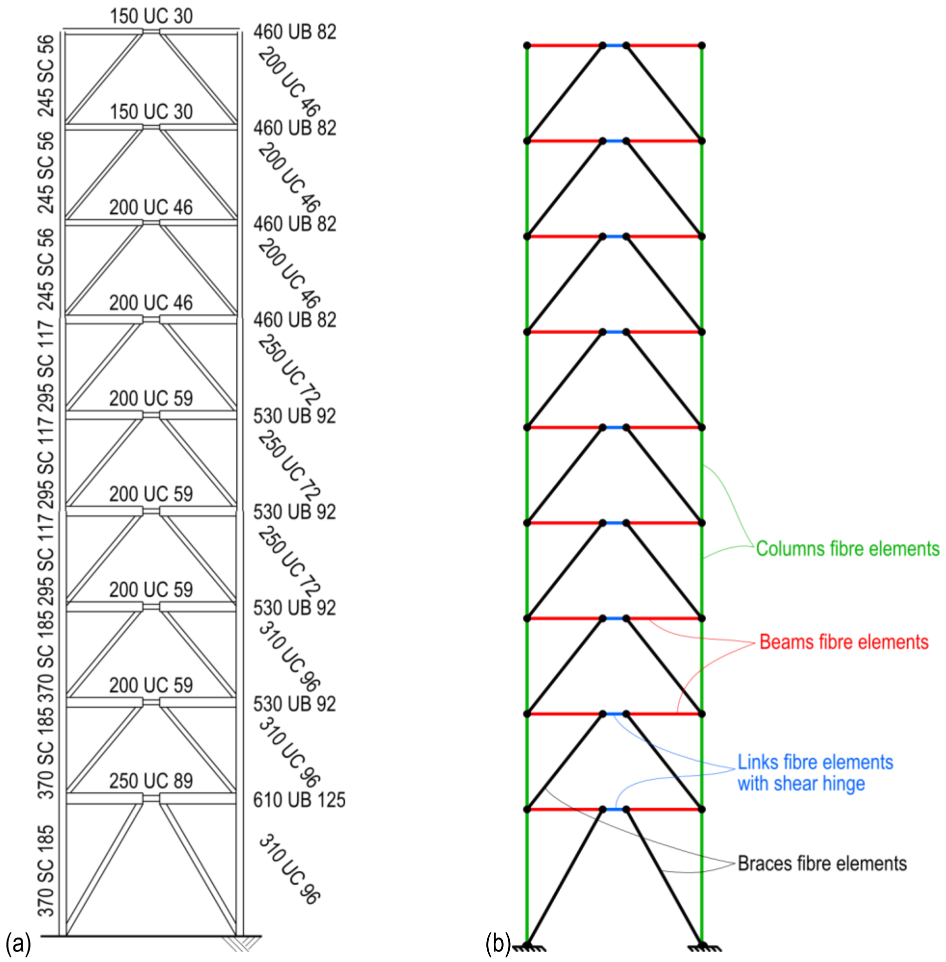


Figure 34. Building G. Sketches of (a) the structural solution and (b) the nonlinear model.

A.7.2 Nonlinear model

The model of Building G is shown in Figure 34b and it consists of a nonlinear frame model where all the structural steel elements are modelled using nonlinear fibre elements. To accurately simulate the inelastic behaviour of the link elements a shear hinge was incorporated in the link fibre elements and calibrated on the structural section's shear strength.

Structural steel G300+ grade was used for the design and analysis of the building and its material properties are summarized in Table 29.

Table 29. Building G. Material properties.

Structural Steel	
Modulus of elasticity, E_s (GPa)	200
Yield strength, f_y (MPa)	300
Post-yielding ratio, r	0.008

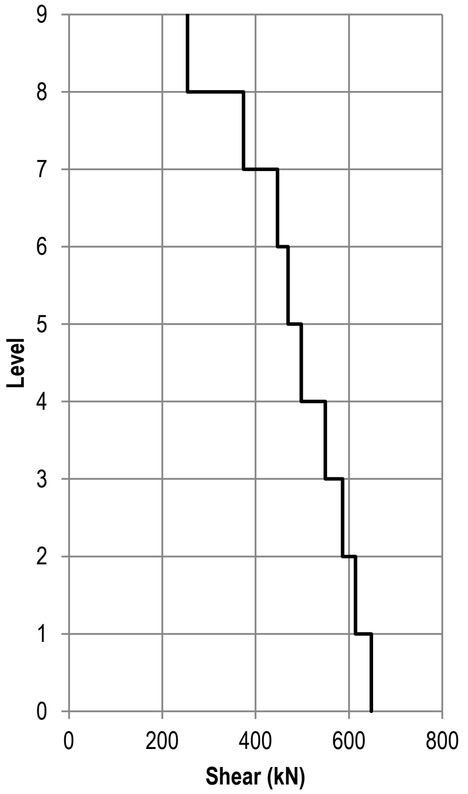
Modal analysis and damping model

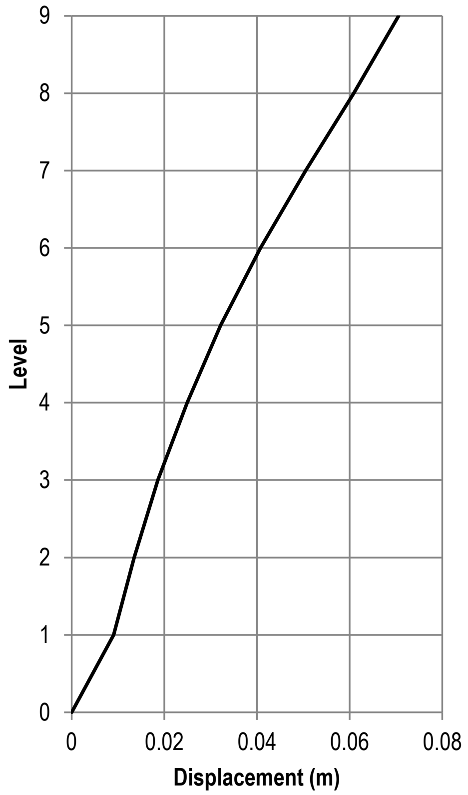
The modal properties of the building are summarized in Table 30. The damping ratios are also reported in the table below.

Table 30. Building G. Modal analysis results.

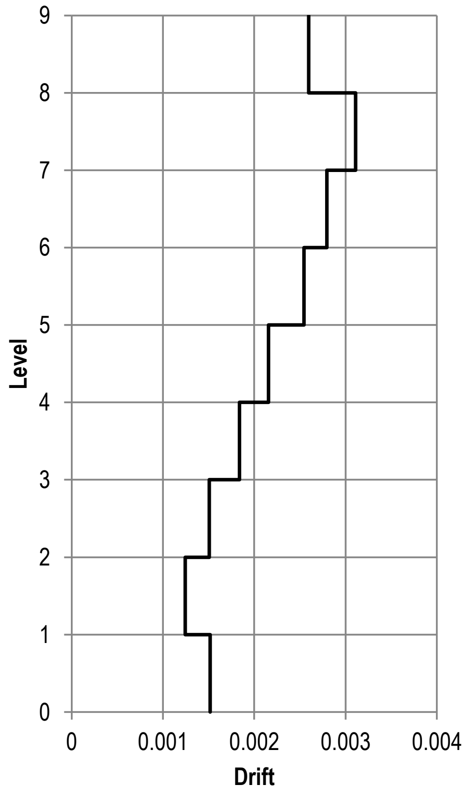
No.	T	Part. Mass		Cum. Part. Mass		Damping ratio
	(s)	(t)	(%)	(t)	(%)	(%)
1	1.792	954	70.4	954	70.4	5.0
2	0.547	311	22.9	1265	93.3	3.6
3	0.286	69	5.1	1334	98.4	5.0
4	0.199	14	1.0	1348	99.4	6.7
5	0.154	5	0.4	1353	99.7	8.4
6	0.129	1	0.1	1355	99.9	9.8
7	0.115	1	0.1	1356	99.9	11.1
8	0.098	0	0.0	1356	100.0	12.8
9	0.087	0	0.0	1356	100.0	14.4

A.7.3 Nonlinear Time-History Analysis results





9
8
7
6
5
4
3
2
1
0
Level
0
0.02
0.04
0.06
0.08
Displacement (m)



9
8
7
6
5
4
3
2
1
0
Level
0
0.001
0.002
0.003
0.004
Drift

Peak Floor Acceleration Study

For MBIE

Report

21 November 2016
106452.04

Holmes Consulting

Holmes Consulting

Level 2, 254 Montreal Street
Christchurch Central
PO Box 6718
Christchurch 8442
holmesgroup.com

Report
Peak Floor Acceleration Study

Prepared For:
MBIE

Date:	21 November 2016
Project No:	106452.04
Revision No:	1

Prepared By:

Jack English

Jack English
DESIGN ENGINEER
Holmes Consulting LP

Reviewed By:

CONTENTS

Holmes

1 INTRODUCTION

Holmes Consulting LP have been engaged by MBIE to assess the distribution of floor accelerations throughout the height of several building types to develop guidance for the design of reinforced concrete diaphragms.

The intent of this study is to investigate the magnitude and distribution of peak floor accelerations (and drifts) obtained from a series of nonlinear time history analyses (NLTHA). The NLTHA will be carried out on three example building types. These include a reinforced concrete (RC) frame building, RC wall building and a steel frame building. NLTHA will be competed on each building type for 3-, 5-, 10- and 15-storeys respectively. The results of this study will be used to examine the current design demands employed for reinforced concrete diaphragm design for each building type.

2 BUILDING DESIGN DATA

2.1 Example Buildings

Three example building structures were developed to base the analysis on. For each building, a typical frame or wall was chosen to complete a two-dimensional NLTHA. Floor inertia demands were introduced into the model via lumped masses at each storey level. As these were two-dimensional frames no diaphragms were physically modelled and only in-plane earthquake and gravity loading was considered.

For each building frame or wall, the tributary area, for determination of inertia forces, was assumed to be 8.0m wide by 15.0m long for the RC buildings and 7.5m wide by 16.0m long for the steel buildings. The analysis models for the RC frames consisted of two bays while three bays were chosen for the steel frame building. Typical storey heights were taken as 3.0m throughout. Building torsional effects and accidental eccentricities were ignored for this study.

2.2 Modelling Assumptions

In order to capture realistic peak floor accelerations and drift profiles up the height of the building, accurate member designs had to be completed. For the RC frame structure, using an Equivalent Static Analysis (ESA), column and beam section sizes were chosen which provided adequate stiffness to meet NZS1170.5 target drift limitation of 2.5%. Once appropriate frame drift was obtained beam reinforcement was determined based on the demands from the ESA. As strength and stiffness are more closely coupled for steel frame sections, further iterations were required to obtain a suitable design.

In order to mimic capacity design principles, columns were kept elastic in the NLTHA models, i.e. no yielding was permitted in the columns with the exception of the base of the ground floor columns.

All RC wall buildings were simply modelled as RC column elements with yielding bases.

2.3 Damping

As typically the case, Raleigh Damping was chosen to approximate the equivalent viscous damping (EVD) for the structure. Damping coefficients were determined based on the fundamental period of the structure, T_1 and a second period, T_N which corresponds to the mode at which at least 90% of the seismic mass has accumulated. For the majority of this analysis, T_N coincided with the third mode of vibration (T_3). For both periods, 5% elastic damping was assumed as per the guidance outlined in the ANSR Manual, Volume 1.

2.4 Design Loading

The seismic loads are derived from NZS1170.5: 2004, Wellington, Soil Class C for a 450-year earthquake, or one which has a probability of occurrence of 10% in 50 years (the normal building life under the code). A summary of the site parameters are shown in Table 1.

Table 1. Site Design Data

Site Subsoil Class	C
Hazard Factor, Z	0.40
Near Fault Factor, N(T,D)	1.00
Importance Level	IL 2
Peak Ground Acceleration, PGA, $C_h(0)$	0.53

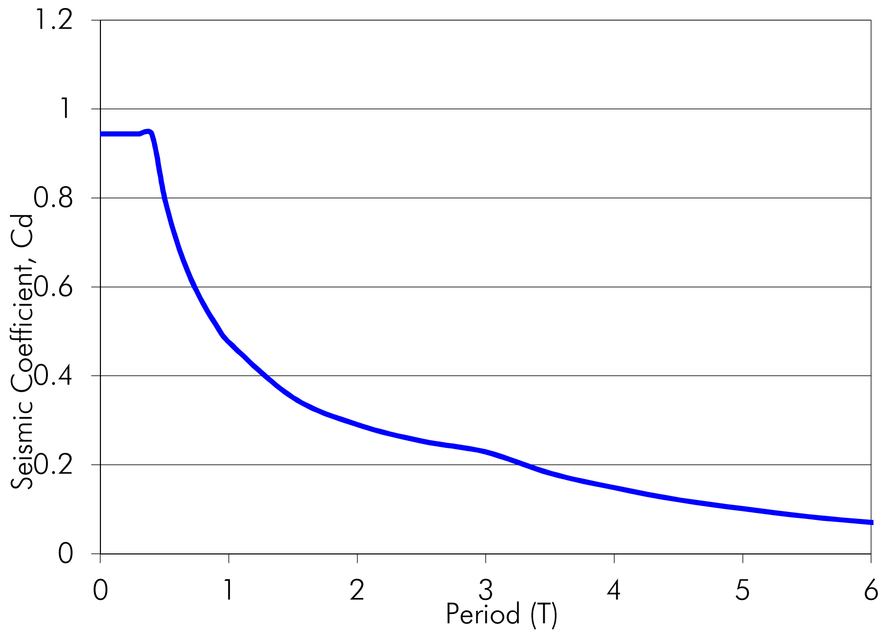


Figure 1. Wellington, Soil Class C Elastic Site Spectra - μ and S_p = 1.0

2.5 Ground Motion Selection and Scaling

Three sets of earthquake ground motions were selected to carry out the NLTHA. As the analysis is only being completed in one direction, only the dominant direction of each record was considered for the analysis. An example envelope of scaled time histories for the three storey RC frame building is shown below.

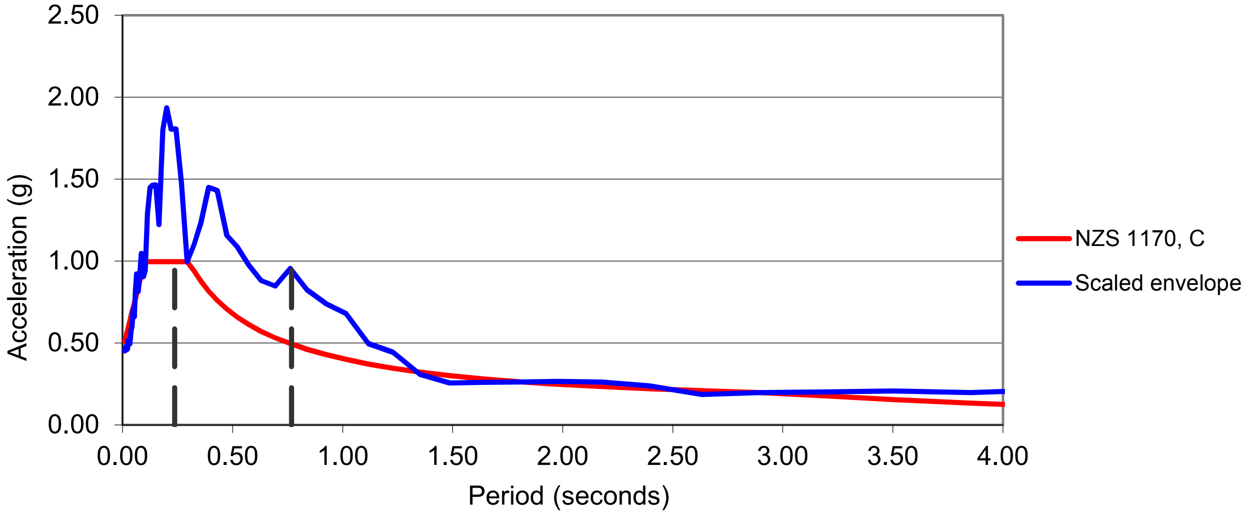


Figure 2. Envelope of scaled time histories for three storey RC frame building

3 ANALYSIS RESULTS

The following sections present the peak floor accelerations (PFAs) and seismic drifts for each building type and for each of the building heights considered as part of this NLTHA study.

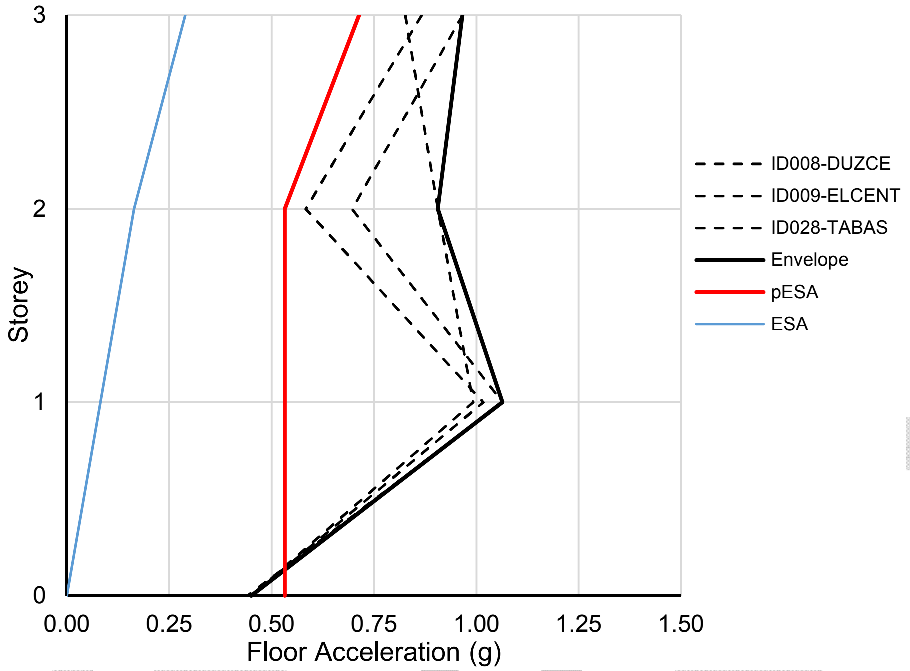
As per NZS1170.5: 2004, the analysis results are taken based on the envelope of three records which have been scaled for the period range of interest. Of note, primarily for the RC buildings, substantial peak floor accelerations have been observed in the lower to middle storeys of the buildings. These results coincide with higher mode effects, primarily mode two, which in most cases falls on the plateau of the acceleration design spectrum and is outside the adopted fundamental period range of interest, where the period range of interest according to NZS1170.5: 2004 is $0.4T_1 \leq T_1 \leq 1.3T_1$ and T_1 is the largest translational period in the direction being considered.

Also plotted against the NLTHA results are the equivalent static analysis (ESA) floor accelerations and the amplified pseudo equivalent static analysis (pESA) floor accelerations for each building. Where, the peak floor accelerations were determined from the ESA by the following equation:

$$\ddot{a} = {}^{F_i}/_{W_i} \qquad \text{Equation 1}$$

And, F_i is the i^{th} storey force determined from the ESA analysis and W_i is the seismic weight of the i^{th} storey.

3.1 Reinforced Concrete Frame Building Results



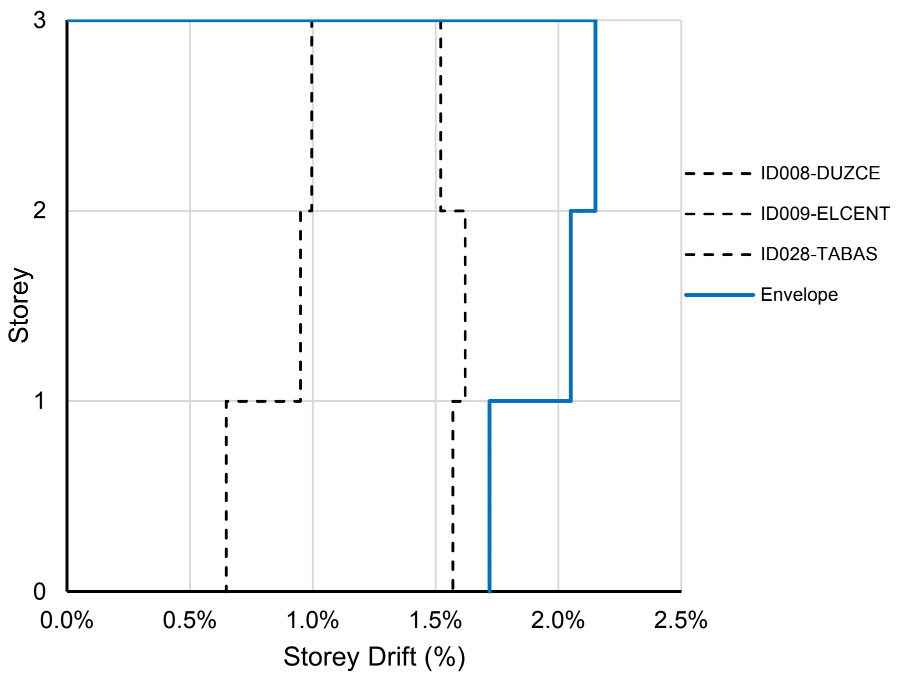


Figure 3. Three storey reinforced concrete frame building – peak floor accelerations and maximum inter-storey drifts

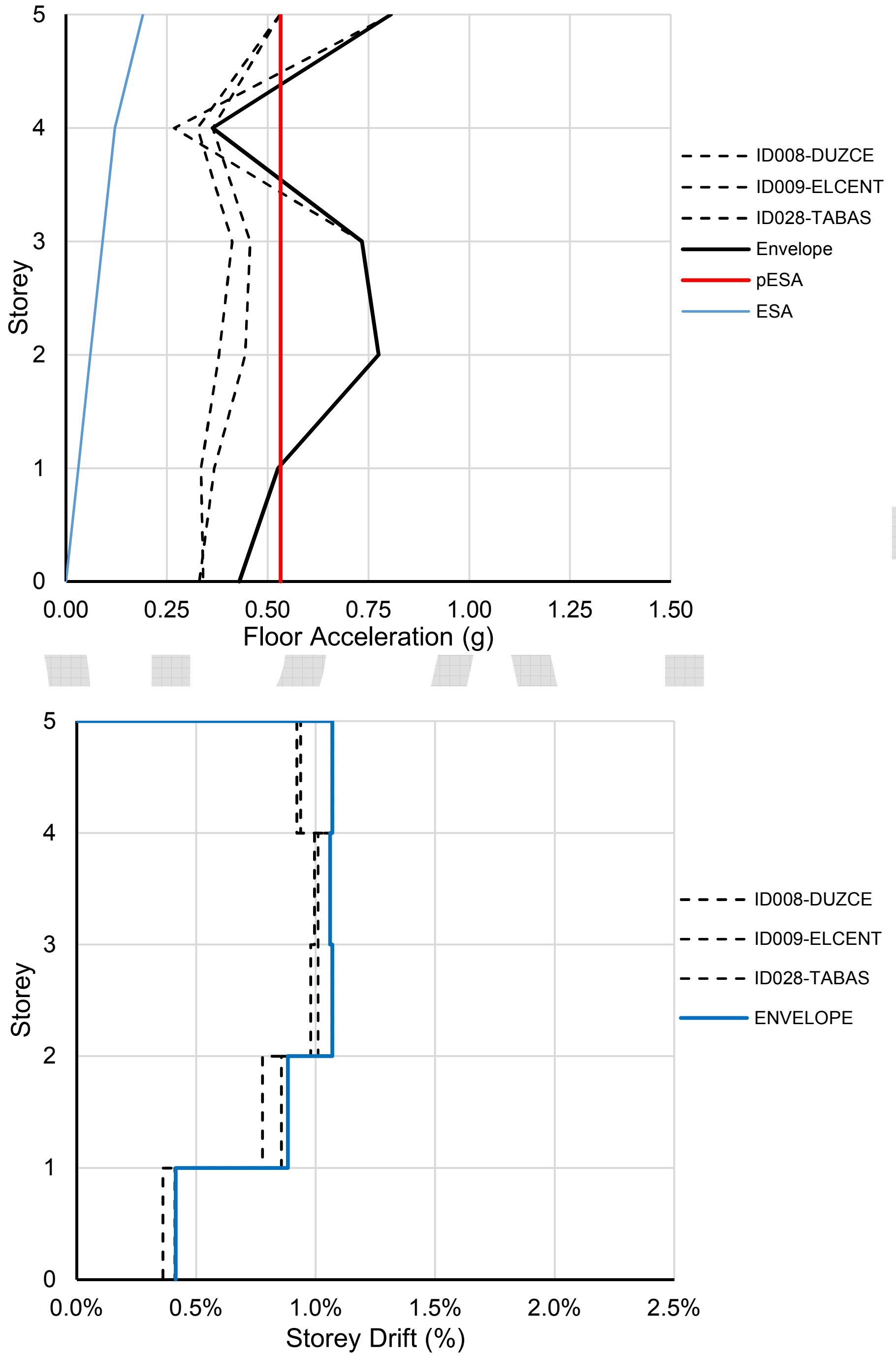
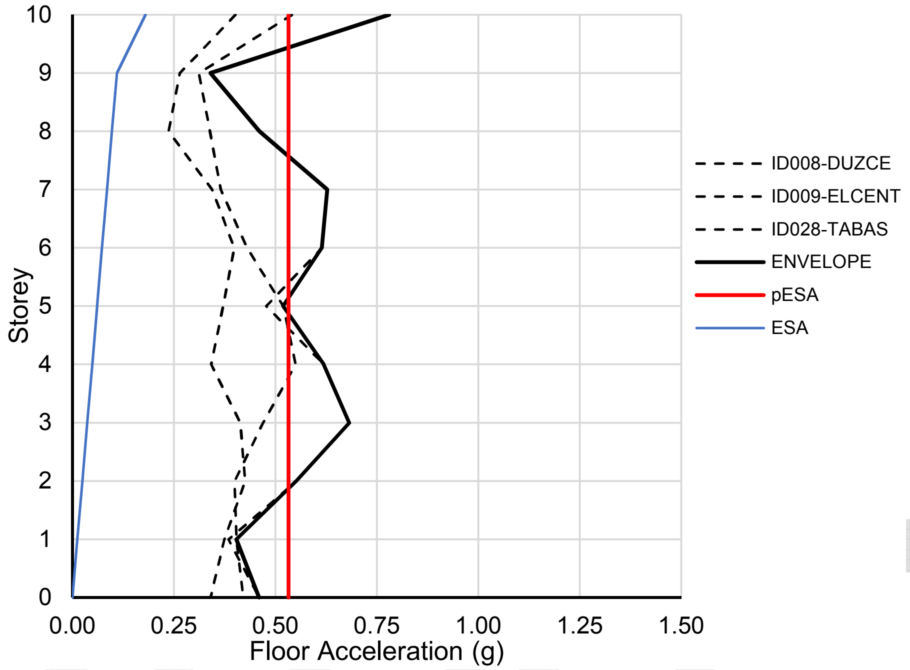


Figure 4. Five storey reinforced concrete frame building - peak floor accelerations and maximum inter-storey drifts



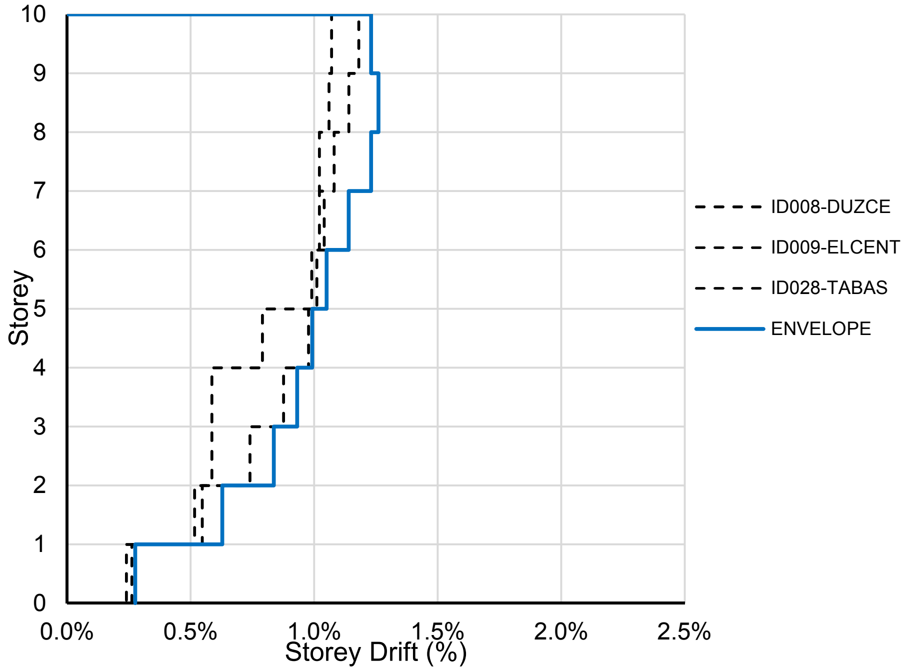
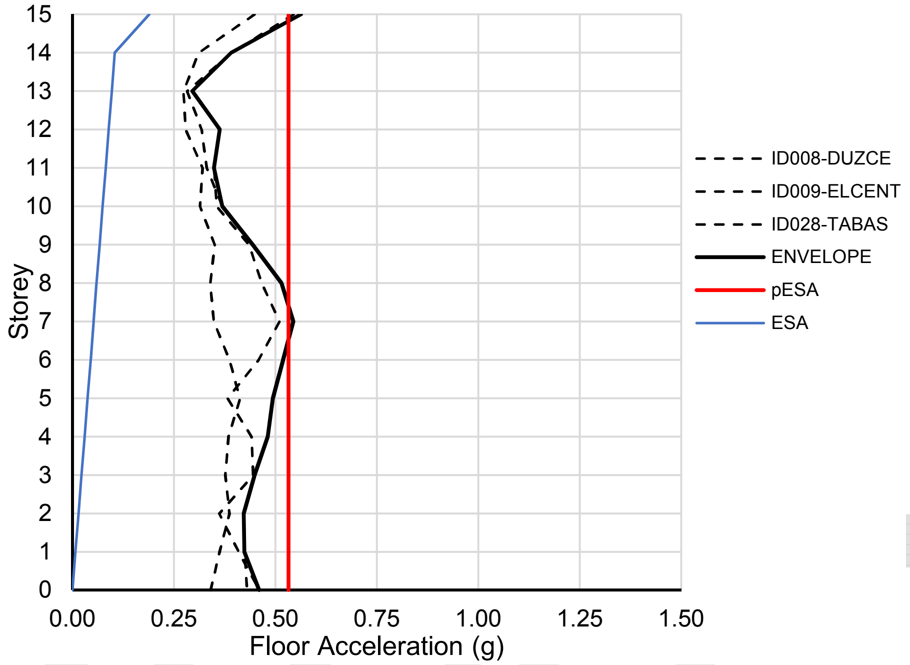


Figure 5. Ten storey reinforced concrete frame building - peak floor accelerations and maximum inter-storey drifts



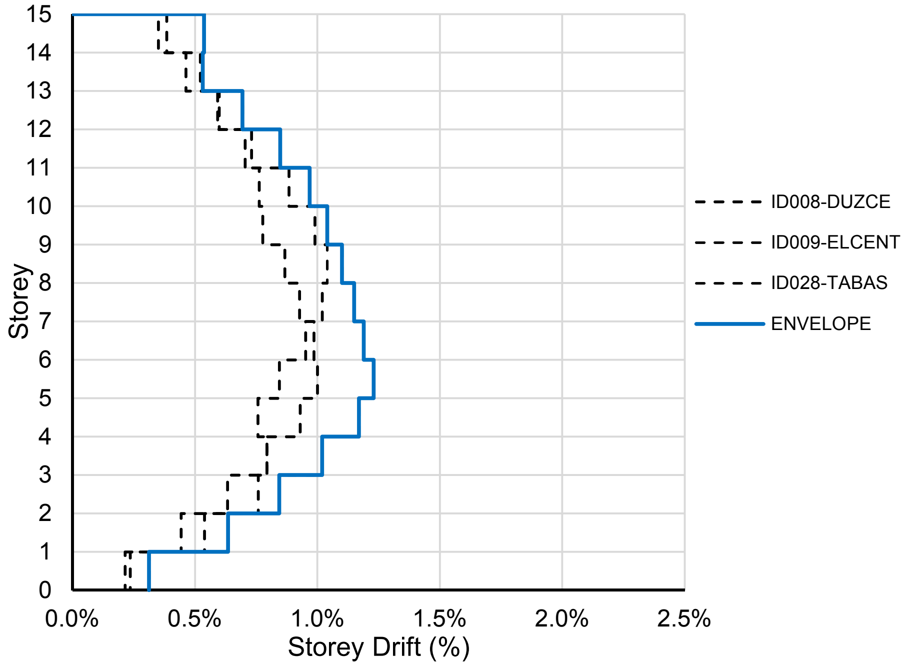


Figure 6. Fifthteen storey reinforced concrete frame building - peak floor accelerations and maximum inter-storey drifts

3.2 Reinforced Concrete Wall Building Results

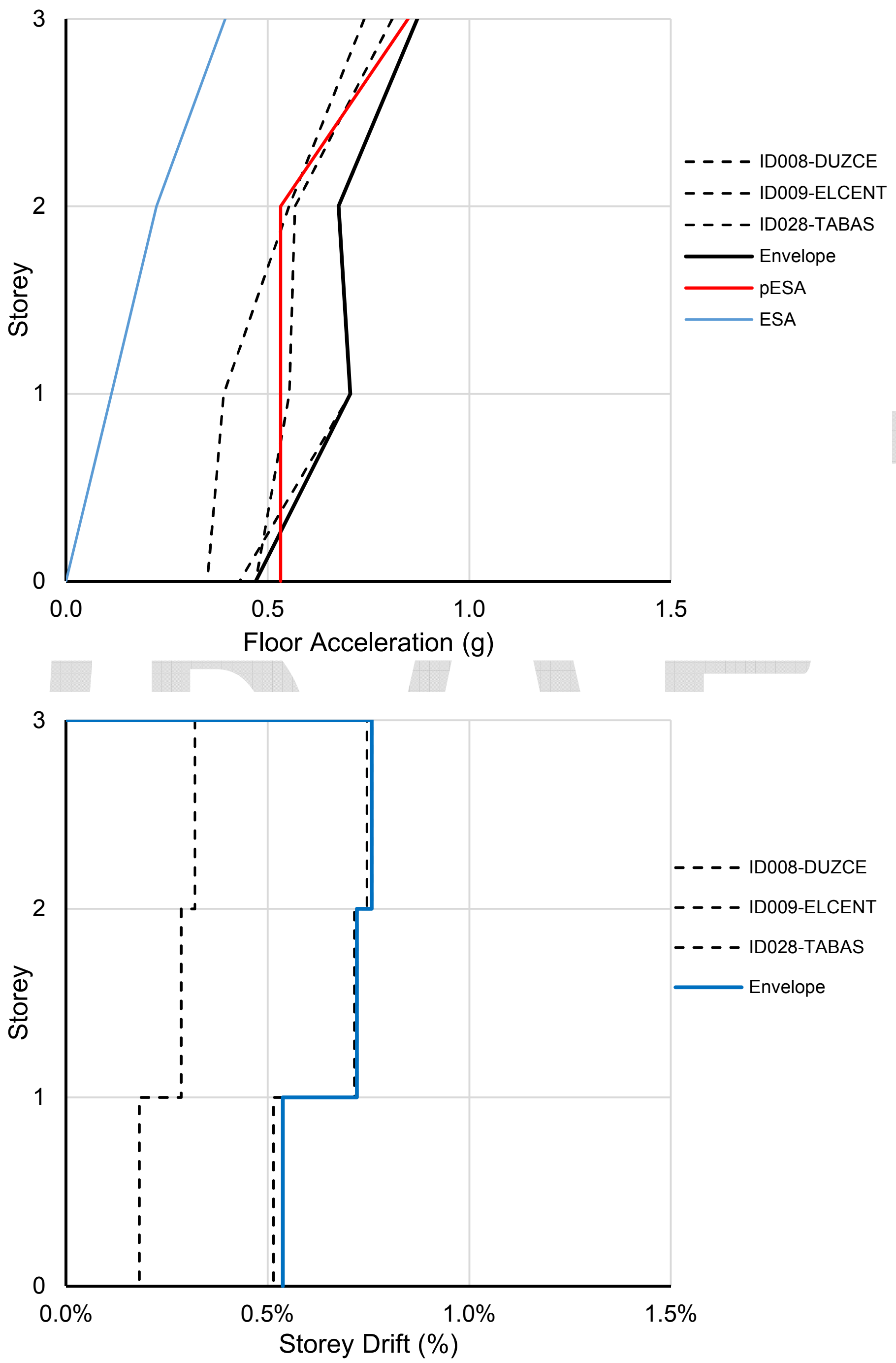
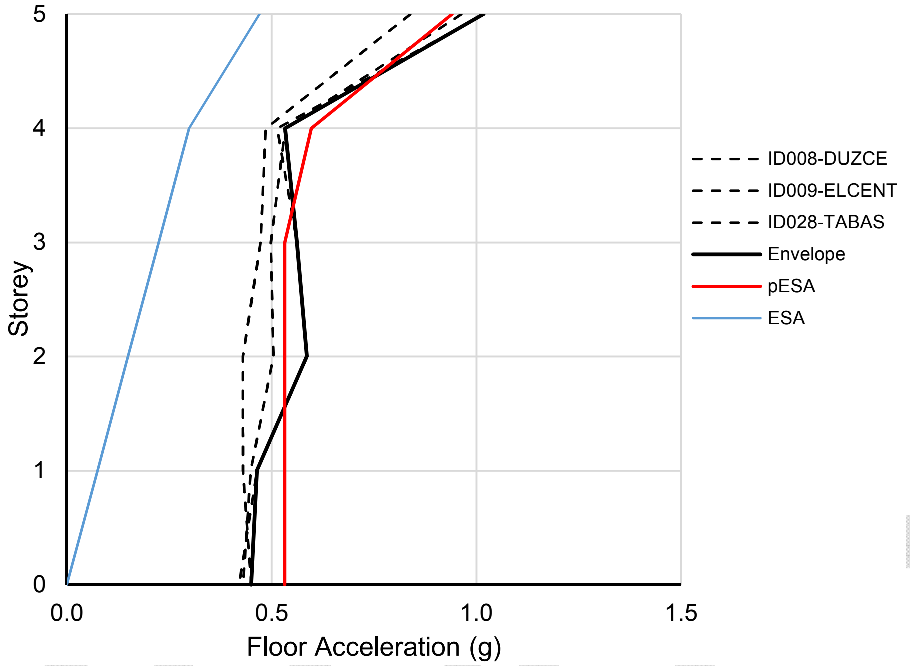


Figure 7. Three storey reinforced concrete wall building - peak floor accelerations and maximum inter-storey drifts



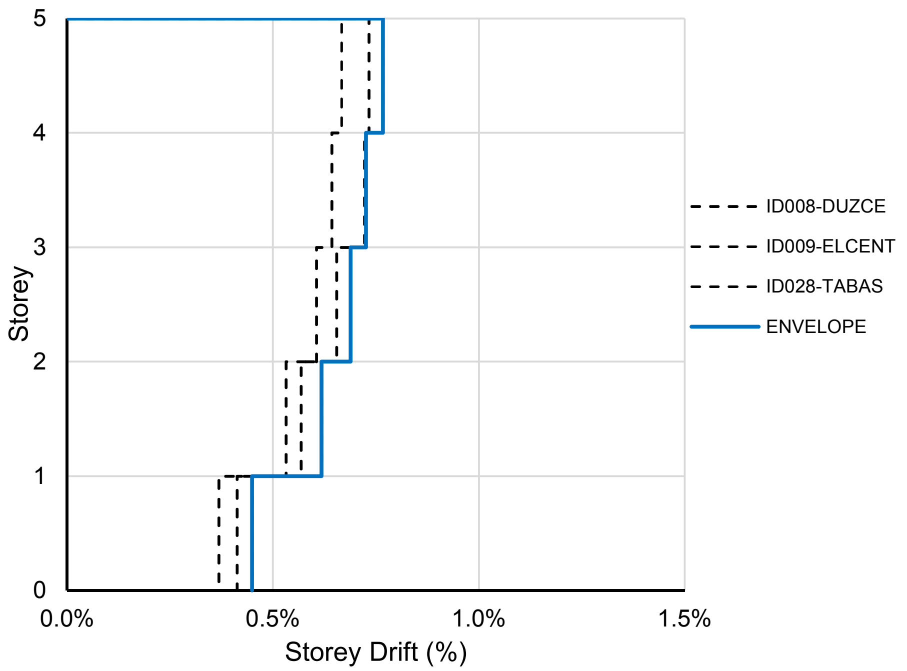


Figure 8. Five storey reinforced concrete wall building - peak floor accelerations and maximum inter-storey drifts

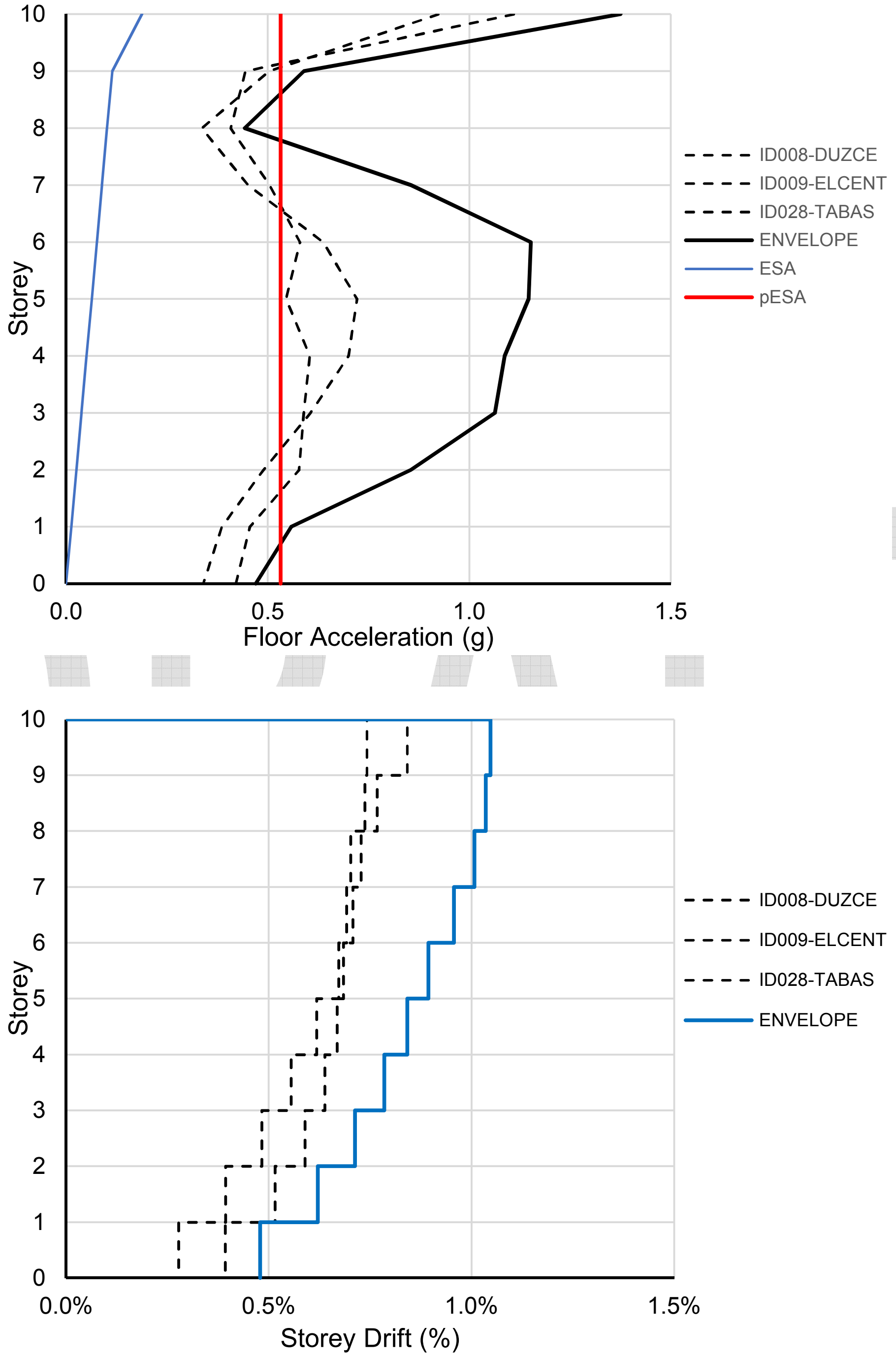


Figure 9. Ten storey reinforced concrete wall building - peak floor accelerations and maximum inter-storey drifts

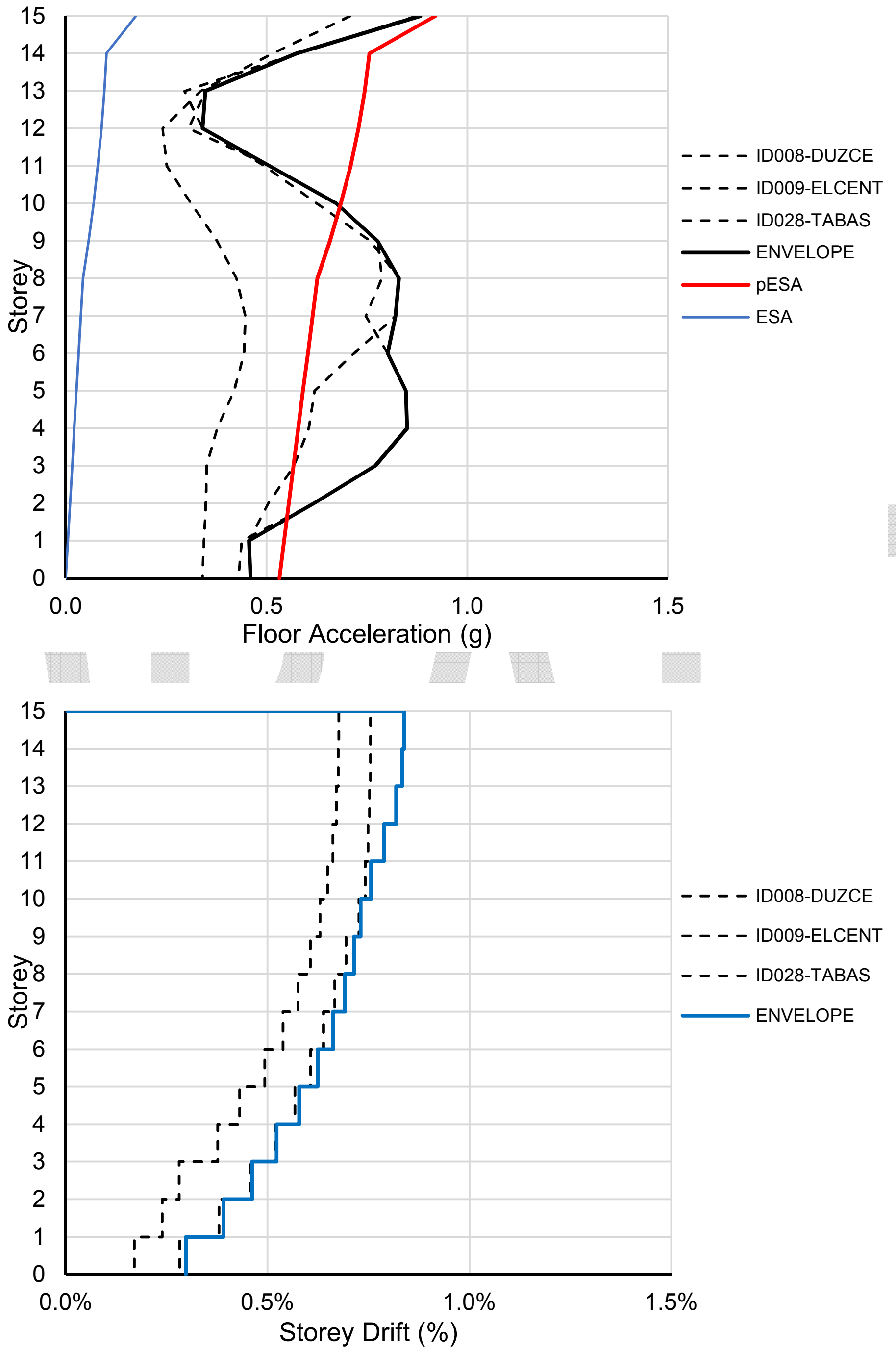


Figure 10. Fifthteen storey reinforced concrete wall building - peak floor accelerations and maximum inter-storey drifts

3.3 Steel Moment Frame Building Results

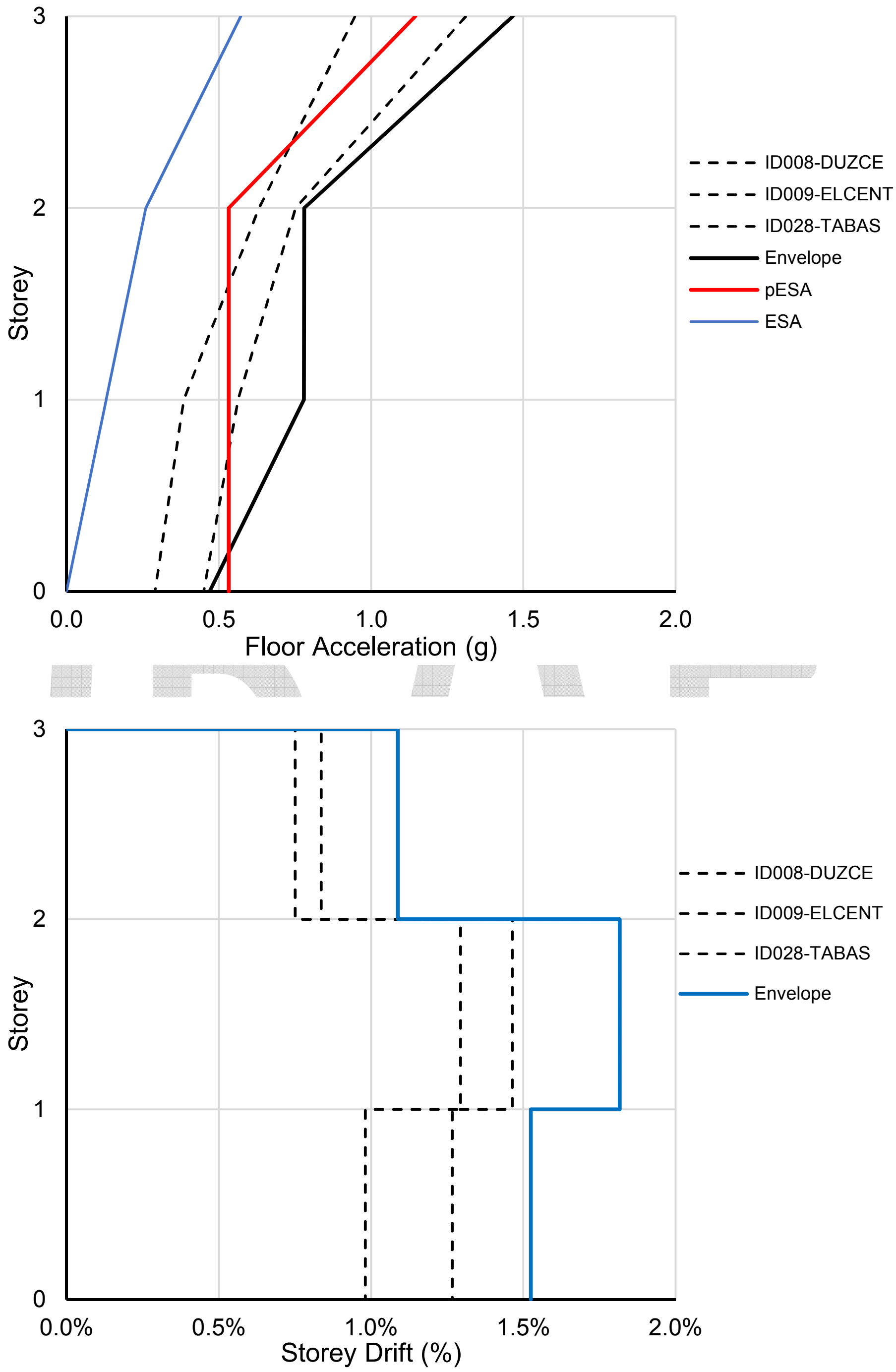
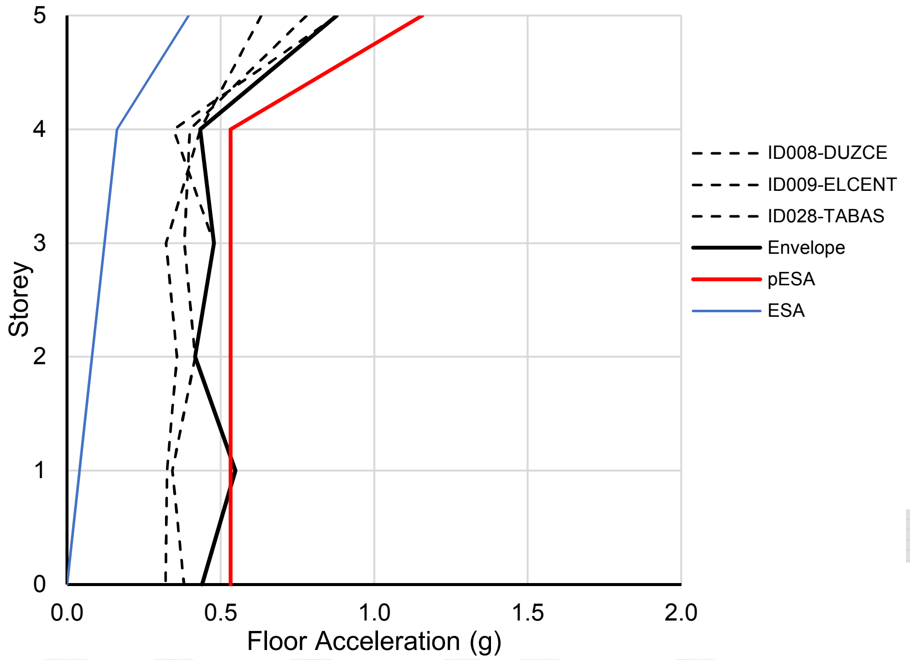


Figure 11. Three storey steel frame building - peak floor accelerations and maximum inter-storey drifts



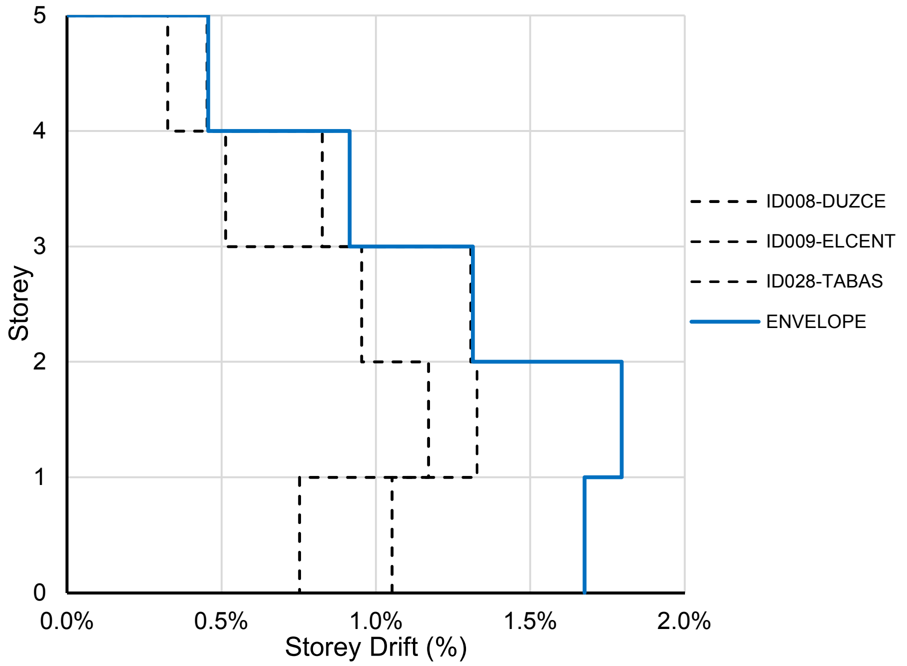
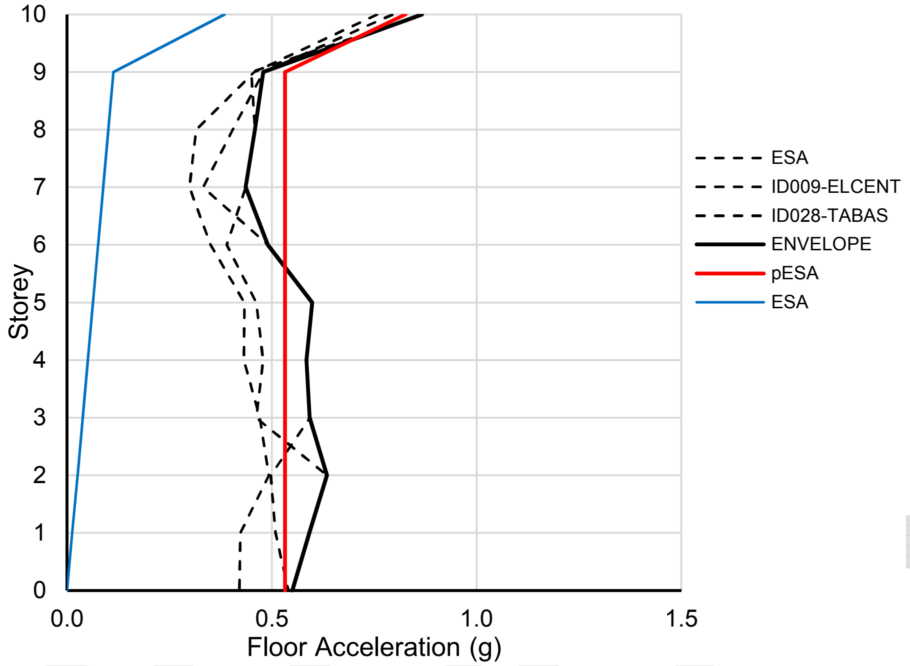
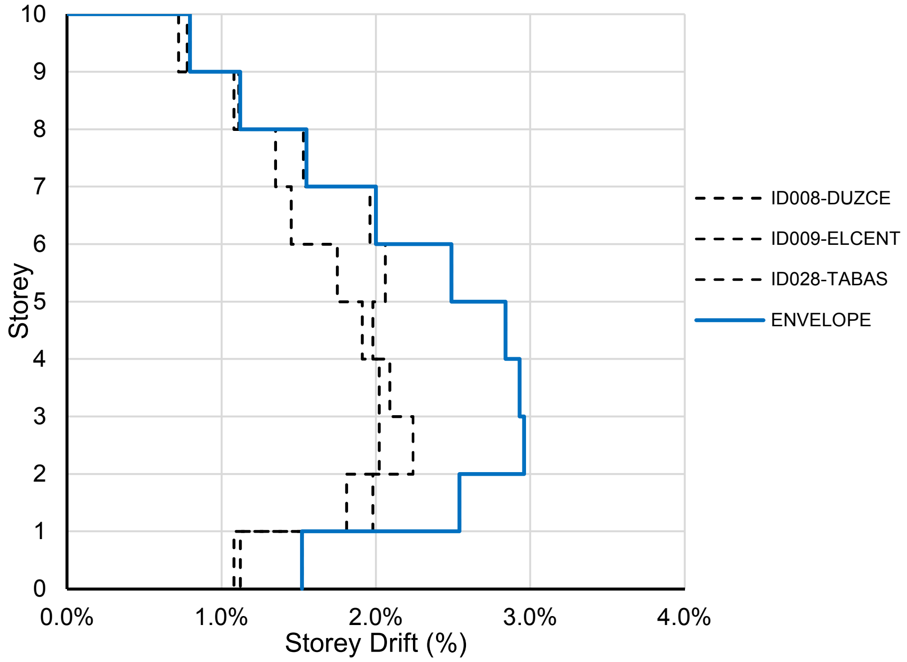


Figure 12. Five storey steel frame building - peak floor accelerations and maximum inter-storey drifts


10
9
8
7
6
5
4
3
2
1
0
Storey
0.0
0.5
1.0
1.5
Floor Acceleration (g)
ESA
ID009-ELCENT
ID028-TABAS
ENVELOPE
pESA
ESA


10
9
8
7
6
5
4
3
2
1
0
Storey
0.0%
1.0%
2.0%
3.0%
4.0%
Storey Drift (%)
ID008-DUZCE
ID009-ELCENT
ID028-TABAS
ENVELOPE

4 LIMITATIONS

Findings presented as a part of this project are for the sole use of the client in its evaluation of the subject properties. The findings are not intended for use by other parties, and may not contain sufficient information for the purposes of other parties or other uses.

Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practicing in this field at this time. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

5 RECOMMENDATIONS

Our recommendations are as follows:

- Instead of taking the envelope of three records as per NZS1170.5: 2004 for the NLTHA, an average of seven or more records would reduce the estimated peak floor accelerations and generally show better agreement against the calculated pESA demands.

- As per the report provided by PTL Structural Consultants dated 04 Nov 2016, a low pass filter could also be applied to each of the NLTHA results by filtering energy content from frequencies higher than a selected cut-off frequency. Refer PLT report for further guidance.