Inelastic response of buildings subject to revised code ground motion

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ABSTRACT: This paper reviews a BRANZ building research levy funded study into the dynamic response of buildings to design earthquakes as prescribed by the draft revised loading standard [SNZ 2000]. This work was the combined effort of Compusoft Engineering (with assistance from Buller George Consulting Engineers), Geological and Nuclear Sciences and the Building Research Association of New Zealand.

The study involved the design and performance assessment of 44 buildings in various New Zealand locations. Variables considered include different ground conditions, high and low seismicity regions, various building heights (3, 10 and 20 storey), different structural materials (steel and concrete), limited and fully ductile detailing, different structural forms (moment resisting frames, shear walls and eccentrically braced frames) and different regularity (regular and torsionally susceptible). The performance of these buildings under three different limit states was assessed.

The paper discusses the procedures used during the study, practical problems encountered, code ambiguities identified and briefly the findings to date. The model buildings that resulted and the various ground motion records selected are available electronically for further study.

1 INTRODUCTION

A draft revision of the earthquake loading standard was circulated for public comment late in 2000 [SNZ 2000]. This resulted in considerable industry effort in preparing over 150 pages of comment on both the earthquake loading provisions, Part 4, and the General Design Requirements, Part 0, which accompanied it.

The draft generated considerable industry comment. The necessity to preserve engineering judgement in the design process was a strong message within the comments. This created difficulties for the Building Industry Authority whose requirements for a Verification Method to be cited in the Approved Documents are that the document be not only clear and correct but also complete and therefore not open to interpretation. Agreement with BIA that structural design procedures could not be prescriptive allowed the redrafting committee to relax some restrictions and to include technical updates and some new provisions into the Standard.

The review committee identified 12 major areas where significant review was needed. The technical effort necessary to resolve these issues was beyond the voluntary contribution of the review committee. Standards New Zealand was asked to form a working group who could investigate and develop a response to the issues raised within these areas.

When considering the way forward, it became apparent that significant benefit could be gained by developing a suite of standard buildings and 'design' ground motions so as to test the implications of alternative solutions.

BRANZ agreed to fund an external contractor to develop a suite of buildings and earthquake ground motion records which could be used to investigate and resolve some of the issues identified. The brief was to design and analyse the inelastic response of a suite of buildings in both high and low seismicity regions of New Zealand. The buildings and ground motion records were to be designed in accordance with the draft earthquake loading requirements with the designs, the models and the ground motions being in the public domain for other studies or comparisons later. The specific objective was to prepare alternative design provisions for the design of building parts when subjected to earthquake attack. The selection of building parts was driven by concern within the review comments about the complexity of these particular provisions. The study provided a basis for developing floor response spectra for buildings as they respond to earthquakes both within and beyond their elastic range.

A review of tenders received confirmed that the effort required to achieve this objective was well beyond the budget available. Because of the importance placed on this study it was, however, possible to reallocate internal BRANZ resources from another research project to supplement the consultant's design effort to achieve the above objective.

2 OUTLINE OF THE BRANZ STUDY

In July 2001 the principal contract was awarded to Compusoft Engineering with Dr Darrin Bell as principal investigator under the direction of Dr Barry Davidson and with Stuart George from Buller George as a nominated subcontractor with specific responsibility for practical design input. A second contract relating to the provision of appropriate ground-motion records and related scale factors was awarded to Geological and Nuclear Sciences (GNS) with Dr Graeme McVerry as the principal investigator. The BRANZ undertook the inelastic time history analysis role.

The Compusoft contract involved the design of 44 buildings which complied with the requirements of the draft earthquake provisions as follows:

- Building sites: a Wellington site with soil class C (Shallow) soil; a Wellington site with soil Category D (Deep or Soft) soil and an Auckland site with soil category C.
- Building configurations: 3, 10 and 20 storey some of which were structurally regular in plan and others structurally irregular;
- Structural Form: Reinforced concrete buildings with moment resisting frames in one direction and shear walls in the other direction; and steel buildings with a combination of moment resisting frames and eccentrically braced frames;
- Both fully ductile and limited ductile buildings considered.

The designs were undertaken using the modal analysis procedures set out in the draft, but with an attempt to meet both the target ductility levels stipulated and the drift limits imposed. The resulting structural models were required to be presented in a form that could be fed directly into the Ruaumoko time-history analysis package. The inelastic time-history analyses themselves were undertaken at BRANZ.

The GNS contract involved selecting appropriate ground motion records for each of the three sites to reflect ground motions associated with the ultimate limit state (ULS) and serviceability limit state (SLS) design events. The response of the building at 0.5 ULS was also of interest in that it was considered that rupture and consequential disruption to non-structural service distributor systems could be deferred to some level between SLS and ULS conditions.

An advisory panel of four consulting engineers with a background in inelastic time-history analyses was formed to advise and monitor the project. The panel consisted of Rob Jury (Beca), Trevor Kelly (Holmes Consulting), Geoff Sidwell (Connell Mott MacDonald) and David Spurr (Spurr Consulting). The panel was tasked to direct the project team to practical solutions that addressed the issues they had collectively encountered when undertaking time-history analyses.

A sub-agenda was to collate this experience as a basis for reviewing the time-history analyses suggested in the standard. Particular attention was focused on the selection and scaling of the ground-motion records and the application of the procedure to design and code compliance.

3 GROUND MOTION SELECTION

3.1 *Steering Committee Direction*

The initial meeting of the project team focused on issues relating to the procedures for selection and scaling of the ground motion records. The key resolutions agreed were:

1. Since the three-dimensional response of the building is of interest, it is essential that the ground-motion records must have at least two horizontal components. In real events these components interrelate. Real, rather than synthetic ground motions were therefore required.
2. The ground motion records selected are to equate with the design spectra used for the modal analysis. The selection procedure required the seismic signature of the record to match (as far as practical) the events that make a significant contribution to the design spectra in the period range of interest. A quality-of-fit check was introduced to achieve a reasonable match.
3. Issues which influence the seismic signature include the magnitude of the event, the physical proximity of the event from the recording device, the slip characteristics of the event and the ground conditions upon which the recording device was located. Later in the study, changes were made to the proposed spectra to give a more gradual fall-off with period and to include near-fault effects. An additional provision was introduced which required that one of every three records was to include a component with marked forward-directivity characteristics when sites are near to the most active major fault systems.
4. The seismic signature varied with the limit state and the location being considered. For serviceability limit state events in Wellington for example, the primary contribution was from distant events with the contribution from the nearby Wellington and Wairarapa faults being minimal. Thus different ground-motion records are necessary for different limit states (i.e. different return periods).
5. There was general agreement that the US practices outlined in the FEMA 273 guidelines (FEMA 1997) and in the Applied Technology Council recommendations, ATC 40 (ATC 1996) should be used as a basis for the selection and scaling procedures of ground motion records for this study. Not all US provisions can be carried across directly, in that for New Zealand the code spectra are defined in terms of the stronger horizontal component rather than the US practice of using the geometric mean of two orthogonal horizontal components.

3.2 *Project Team Application*

Discussion regarding the detail of the ground motion scaling procedure followed over several weeks but it was eventually agreed that:

1. As with the practice in the USA, a period range of interest needs to be established in order to match the actual record with the design spectra. Both short-period response and the potential for the building to soften and move well beyond its assessed fundamental period need to be considered. The period range of interest used in the USA is $0.2T<T<1.5T$ (where T = the fundamental building period). Although the basis of selecting this particular range is not clear, it appears to cover the above requirements and was used for this study also.
2. A minimum of three ground-motion records are required. Each record is to have a

similar seismic signature to the significant events contributing to the design spectra at the target period and for the return period associated with the limit state being considered.

3. The accelerations in each record are scaled by two factors. First, the record scale factor, k_1 adjusts the record to match the design spectrum as closely as practical over the period range of interest. The component with the lowest scale factor at each period was identified as the principal component for that period. Second, a family scale factor, k_2, magnifies all records so as to ensure that at least one record exceeds the target or design spectrum at each point over the period range of interest.

The presentational style offered by GNS resulted in a set of period-dependent record scale factors each of which identified the principal component of that record at each period. An example of the form presented is shown in Figure 1

				Scaling factors		RMS1=		RMS2=	
T(fit)	Tmin	Tmax	Nperiods	Kfirst=	Ksecond=	rms log(error1)	10 ^RMS1	rms log(error2)	10 ^RMS2
0.3	0.06	0.45	41	2.89	1.87	0.069	1.17	0.053	1.13
0.5	0.10	0.75	45	2.58	1.71	0.074	1.18	0.059	1.14
1.0	0.20	1.50	45	2.60	1.98	0.065	1.16	0.126	1.34
1.5	0.30	2.25	42	2.70	2.28	0.083	1.21	0.128	1.34
2.0	0.40	3.00	40	2.86	2.53	0.074	1.18	0.124	1.33
2.5	0.50	3.75	38	2.87	2.84	0.064	1.16	0.140	1.38
3.0	0.60	4.50	38	2.92	3.37	0.055	1.13	0.176	1.50
4.0	0.80	6.00	36	2.90	4.02	0.058	1.14	0.176	1.50
6.0	1.20	9.00	33	2.75	4.66	0.061	1.15	0.133	1.36

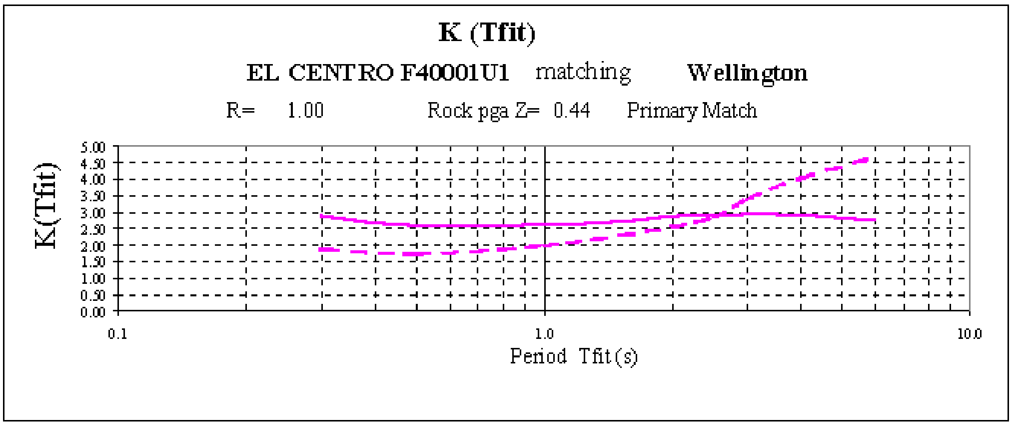

Figure 1 Record Scale Factor (El Centro Record - Wellington Deep Soil Application)

Records for the different limit states at each site together with their record scale factors were selected and provided to the project team by GNS. The target spectrum is derived in accordance with the requirements of equation 3.1 of the draft earthquake loading standard (SNZ 2000) modified to include the more recently introduced near fault factor, N(T,D). Thus the target spectrum is the product of the spectral shape factor, $C_H(T)$, the zone factor for the site, Z, the return-period factor, R, and the Near-Fault factor. An additional requirement was that at least one record within each family exceed the target spectrum over the period range of interest. This is achieved by achieved by applying the family scale factor, k_2 (Refer Figure 2). The envelope approach is slightly different from that used in US practice where the average spectral ordinate of the set of records is required to exceed the target spectra. However since the most adverse response of a parameter to each member of the family controls acceptability, it is considered reasonable that only one record need exceed the target. An example of the comparison of the target spectra to the family of principal spectra for a T=1.5 second building on Class D soil in Wellington under Ultimate Limit States condition is shown in Figure 2. The envelope of the three spectra falls below the target spectrum near the maximum period of interest, 2.25s for a 1.5s structure, requiring a k_2 factor of 1.13.

Unfortunately the above procedures where subjected to review and the proposed code spectra modified as the project progressed and unexpected issues arose. This resulted in considerable rework by both GNS selecting the records and record scale factor and by BRANZ who were determining the family scale factors and applying them to the building models to ascertain the inelastic response parameters. The agreed procedure (as outlined above) has been offered to the standards review committee as the basis for the Time History Analyses procedure described in Section 6.4 of the revised draft earthquake loading standard.

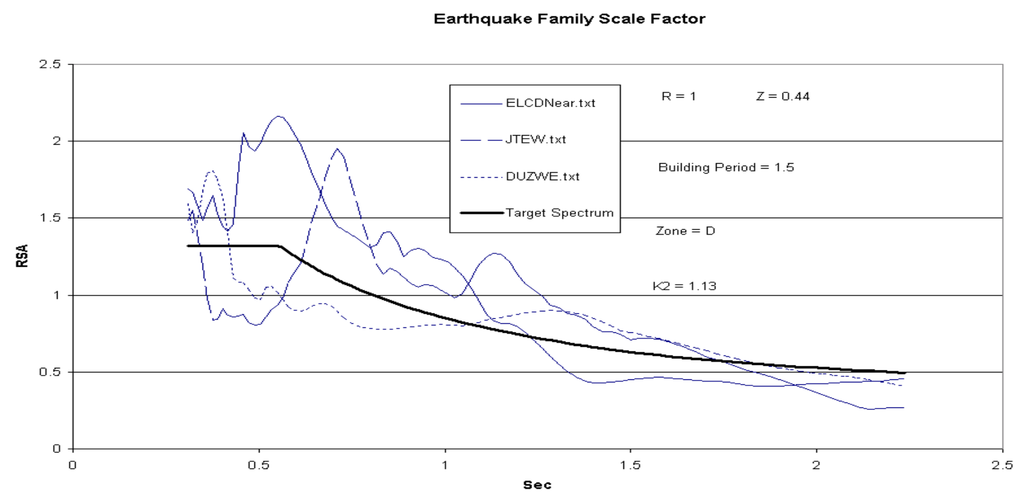

Figure 2 Typical Family Scale factor, k_2 for T=1.5, Wellington, Class D, ULS

4 DESIGN AND MODAL ANALYSES

Compusoft Engineering, in conjunction with Buller George, consulting engineers, were tasked with the design of the buildings indicated in Table 1. The designs were to comply with the provisions outlined in the draft earthquake loading provisions (SNZ 2000) and were to comply with the constraints imposed by the various material standards.

4.1 *Design Constraints*

To constrain the number of designs and models required the following design constraints were considered by the steering committee and imposed:

- All buildings are to be rectangular in plan (6 bays by 4 bays) with moment resisting frames (MRFs) in the longitudinal or z axis direction and either shear walls (for the concrete buildings) or eccentrically braced frames (EBFs) in the shorter transverse or x axis direction.
- The column grid layout used for the steel building was different from that used for the concrete building reflecting the more common design practice as expressed by the advisory panel.
- The number of floors (N in Table 1) represents the number of occupied floors with a roof structure above the upper floor for modeling purposes.

Wellington Intermediate

	Reinforced concrete		Steel MRF/EBF	
	mu=3	mu=6	mu=3	mu=6
Plan Reg				
N=3	X		X	
N=10	X	X	X	X
N=20	X	X		
Plan Irreg				
N=3	X		X	
N=10	X	X	X	
N=20	X	X		

Auckland Intermediate

	Reinforced concrete		Steel MRF/EBF	
	mu=3	mu=6	mu=3	mu=6
Plan Reg				
N=3	X		X	
N=10	X	X	X	
N=20	X	X		
Plan Irreg				
N=3	X		X	
N=10	X	X	X	
N=20	X	X		

Wellington Soft

	Reinforced concrete		Steel MRF/EBF	
	mu=3	mu=6	mu=3	mu=6
Plan Reg				
N=3	X		X	
N=10	X	X	X	X
N=20	X	X		
Plan Irreg				
N=3	X		X	
N=10	X	X	X	
N=20	X	X		

No Building Models = 10
No Building Designs = 44
Modal Analysis to 2000 Draft Spectra

Table 1 Building Design Matrix

- The plan irregularity was introduced by altering the location of one or more of the transverse lateral load resisting elements.
- The interstorey height was 3.65 m for all but the first floor where a height of 4.5 m applied.
- Column bases were considered to be partially fixed to allow for some foundation relaxation.Variations in beam depth, beam reinforcing and column reinforcing were constrained so that a maximum of three changes occurred within the height of each building.
- Beam column rigid junction zones were considered to extend over half the column width.
- In all cases the floor systems were assumed to be prestressed concrete floor panels with a concrete topping. These were considered to span parallel to the longitudinal frames and to be supported by a gravity load carrying frame which did not contribute to the lateral load resisting system.
- The designs of the concrete frames were adjusted so that the stiffness of the framed met the deflection criteria of Part 8 of the draft standardas closely as possible, including allowance for P-delta (where appropriate) and the strength was the minimum needed to satisfy the compliance requirements (ie there was minimal reserve capacity) of the standard.
- The design of the steel buildings was controlled by the interstorey deflection limits, with the base shear being as close as possible to the values designated in the draft standard within the constraints that commonly available steel sections were to be used.

4.2 *Models for Analyses*

ETABS 7 was used as the modal analysis package. A three dimensional model of each building was developed and their response to the draft earthquake spectra assessed to determine beam actions. Capacity design was applied to determine column actions. Rigid floor diaphragms were assumed at each floor and a dummy column established at which the floor mass was located. Accidental eccentricity was introduced by relocating the dummy column. P-Delta effects were included both within the modal analysis and the time history models. This feature could be initiated as required.

4.3 *Design Implications Arising from Draft Procedure*

As had been anticipated, there was an early conflict between attaining the structural ductility targets and complying with material standard constraints. In particular it was not possible to attain the high ductility levels in Auckland and satisfy the minimum steel content within the concrete structures.

Several issues arose during the design process and are discussed in more detail in the companion paper (Bell et al 2002). Of particular interest however is the apparently much greater contribution of higher mode effects to the base lateral shear. This is an inevitable result of the much larger difference between the short (<0.4 sec) and intermediate (>1 sec) spectral ordinates proposed (increasing from around 3 in NZS 4203 to around 8 in the 2000 draft). This anomaly was brought to the standard review committee's attention and has resulted in a review of the elastic spectra and a diminishing of the difference. For equivalent static analyses, a proposal is being considered to superimpose a higher mode contribution at the upper level of the building in addition to the triangulated distribution of the first mode assessed base shear. The effect would be to impose a similar level of base shear on buildings analyzed by either modal or equivalent static design procedure. Other implications are however still being considered.

5 INELASTIC TIME HISTORY ANALYSES

One of the objectives of the project was to provide model buildings that can be analysed for purposes beyond the scope of this study. It was therefore considered important that the analysis package selected to undertake the inelastic analyses was commercially available, reasonably priced, and easily accessible. The package was required to be capable of three dimensional analyses and be able to support a variety of hysteretic degradation models. Ruaumoko 3-D (Carr 2001) appeared to fulfill this specification and was selected as the analyses software. This decision caused some problems when the originally supplied version of the software resulted in the model becoming unstable. However, modifications to the program overcame this difficulty and the results now being obtained appear reasonable. A check was made later in the project with three of the frames being reanalyzed using the ANSR analysis suite and the results were consistent with those obtained using Ruaumoko.

The inelastic models used the ideal member strengths since the overall target remained the derivation of floor response accelerations from which the design of building parts could be assessed. The modified Rayleigh damping was used do for the analyses

5.1 *Analysis Procedure*

Each model was first subjected to two pushover analyses firstly with the action being imposed in the longitudinal frame direction and secondly in the transverse direction. The first eight response periods were established and plotted. The roof displacement to base shear plot was used to establish the structural ductility curves for each building. The maximum building deformation envelope and the deformation profiles for the building at maximum deformation and at maximum interstorey drift were developed and plotted for both horizontal axes.

With the complexity of the various models and the potential for error, there was a strong preference for using the Ruaumoko batch file entry option. An Excel spreadsheet was configured in such a manner that a simple reference to the building identifier resulted in the generation of a batch file which could be read directly into Ruaumoko. This proved to be a very efficient means of generating each model and avoided the error potential which would otherwise have been expected for the 880 nodes and 987 members of the 20 storey, reinforced concrete building model.

Within each spreadsheet, one for the steel and a second for the concrete building models, a front page was created for each of the 3, 10 and 20 storey buildings. These contained all of the data necessary to describe to Ruaumoko the building nodes, its members and their properties, and the excitation to be applied to the structure. Different cells were indexed to other sheets in the workbook, and items such as some of the member properties would change based upon the building properties selected i.e. regular or irregular plan, location within New Zealand, soil type and ductility. A macro created the text file in a form that was suitable for input into Ruaumoko and linked to the text files containing the two components of the ground motion record.

The text files created as described above were used as batch file input for each Ruaumoko run by batch file. These were set up to run overnight with processing time of between 10 minutes (3 storey) and 2 hours (20 storey) being achieved on the 1 GHz processor PC.

The application of the ground motion to the building requires that the principal component of each record be applied firstly in longitudinal building orientation (with the secondary component in the transverse direction). Both components are required to be scaled by the record scale factor applicable to the principal component and the family scale factor applicable for the fundamental period of the building in the orientation of application. The building is then re-orientated and the procedure repeated with the target period now being that of the building response in the transverse direction. Thus, since each building is required to be considered under at least three ground motion records, at total of 6 runs for each building for each limit state is required.

5.2 *Post Analysis Processing*

To standardize, a suite of interrogation programs were developed at BRANZ to extract data from the Ruaumoko output files. The specific applications envisaged included the extraction of the following:

- Master and four corner node x and z interstorey drifts and the time interval at which they occurred.
- Internal actions at the start/finish of selected members (later processed into the moment curvature results of those members).
- Mode orientations, shapes and periods for the first 12 response modes of the structure (used primarily during the pushover analyses)
- Master node transverse and rotational accelerations at each time step of the analysis (which were later used to determine floor response spectra and displaced shapes),

5.3 *Application of resulting output*

Typical output from the analysis of a specific building under a given ground motion is provided in Figure 3. The floor response spectra are presented at three floor elevations up the building with the response at two corners and the master node at each floor being used as input into the floor response spectral derivation.

The overall and interstorey drift plots include deformation in the x direction (RHS) and the z direction (LHS). The deformation envelope, and the deformed shape at the instant of maximum roof displacement and at the instant of maximum interstorey drift are plotted. The results are also to be used to assess the comparison between the scaled modal deformations and the inelastic response (indicative comparisons are provided in Figure 4. For comparison purposes the inelastic deformation plots were prepared using record scale factors which matched the design spectrum in the draft. The 1.5% limit applicable for modal analysis appears to result in building deformations of around the 2.5% limit imposed for inelastic analysis.

Since it is the response of the building under the family of records which is to control the response parameter under consideration, the envelope of floor response spectra for a particular building subjected to each of the three earthquakes in two orthogonal directions has been used in deriving the design requirements for Parts (Shelton et al 2002).

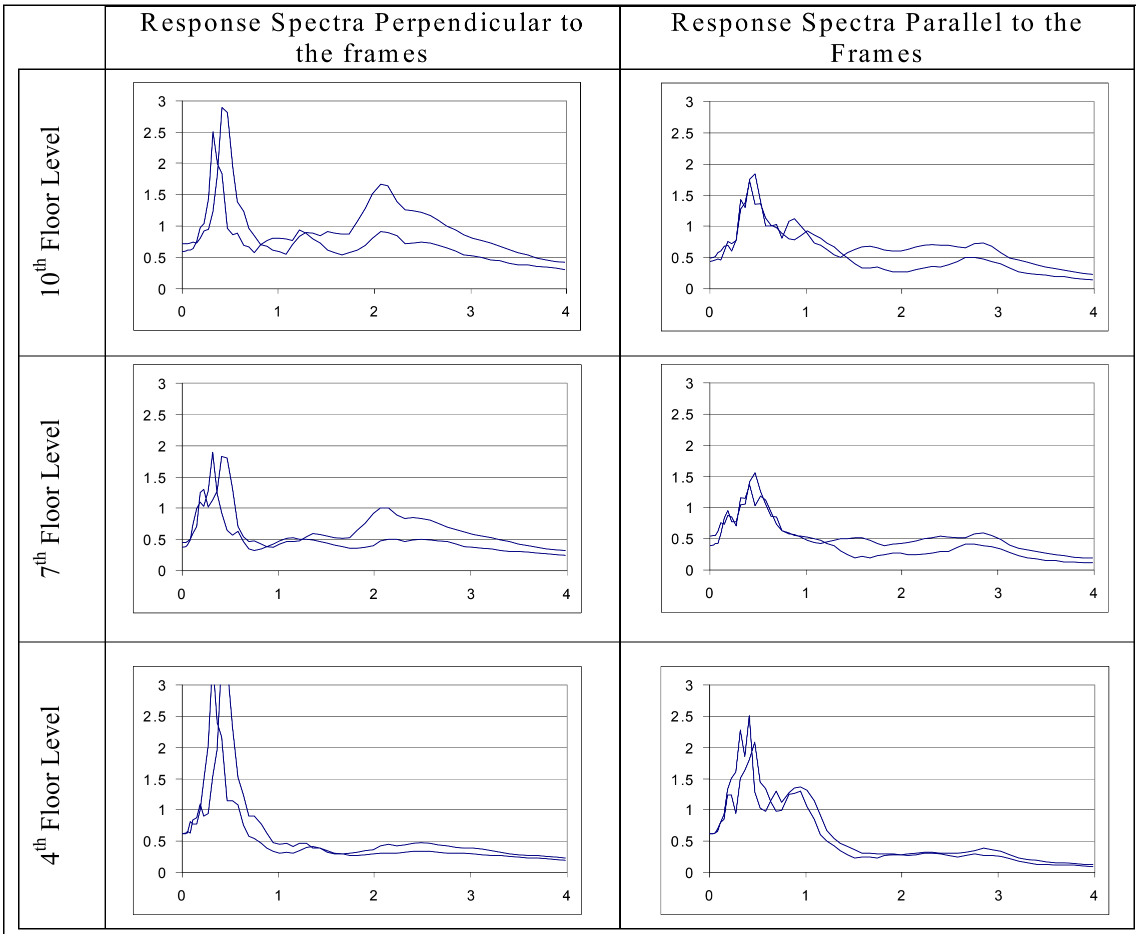

Figure 3 Inelastic Time History output – Floor response and deformed profile

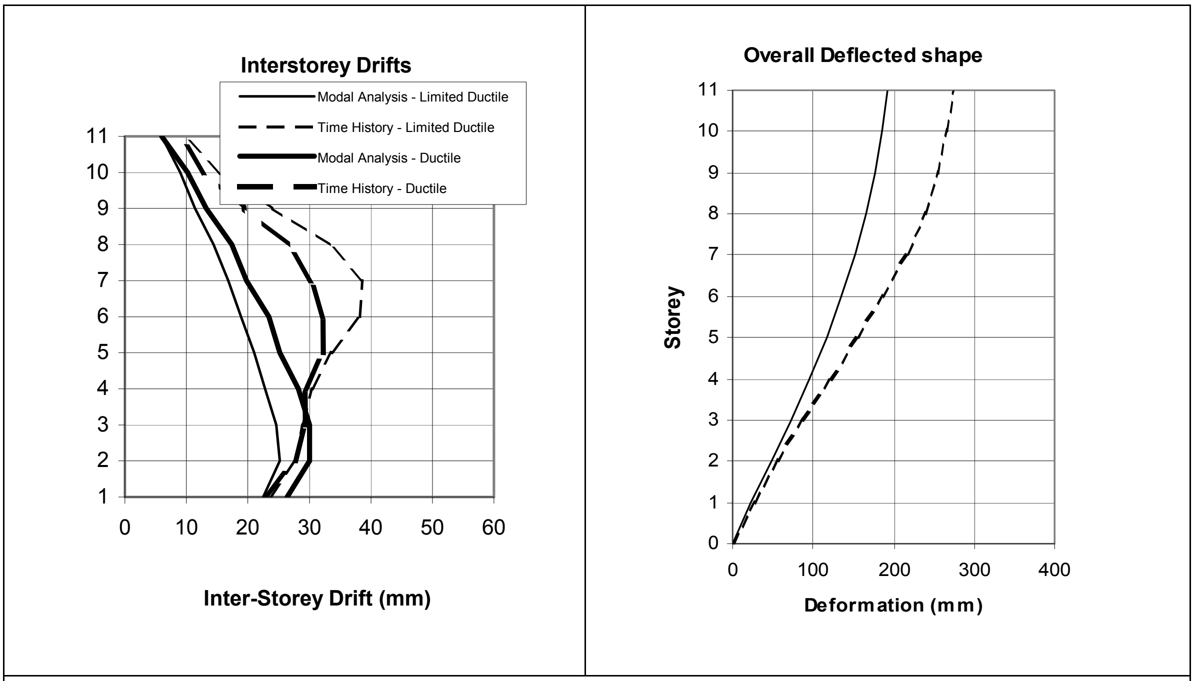

Figure 4 Drift comparisons (Inter-storey LHS) and overall (RHS)

6 CONCLUSION

The study has provided extremely valuable input to the development of both the inelastic time history procedures to be applied to the revised earthquake loading standard draft and as the basis for the complete revision of the design of building parts.

There have also been direct applications to the revision in the following areas:

- The proposed spectral shapes for time-history and modal analysis have been revised.
- The equivalent static design spectra have been reviewed and a short period truncation reintroduced.
- The greater contribution of higher mode effects as a result of the spectra shape changes from NZS4203:1992 has been identified and changes to the means by which these effects are to be considered in the equivalent static procedure are being considered.
- The means of calculating post elastic deformation from either modal or equivalent static design procedures is under review with a comparison between these methods over various building configurations continuing.

In addition, BRANZ now has a set of code-compliant building designs (designed in accordance with the 2000 draft spectra) available for further work. The ground-motion records and scale factors applicable to the two Wellington sites and the Auckland site are also in hand. Both are to be made available to the public either for further research applications or for direct application to time-history analyses.

7 ACKNOWLEDGEMENTS

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