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Role of residual displacements in performance-based seismic assessment, design and retrofit of reinforced concrete buildings and bridge structures: Assessment and mitigation strategies

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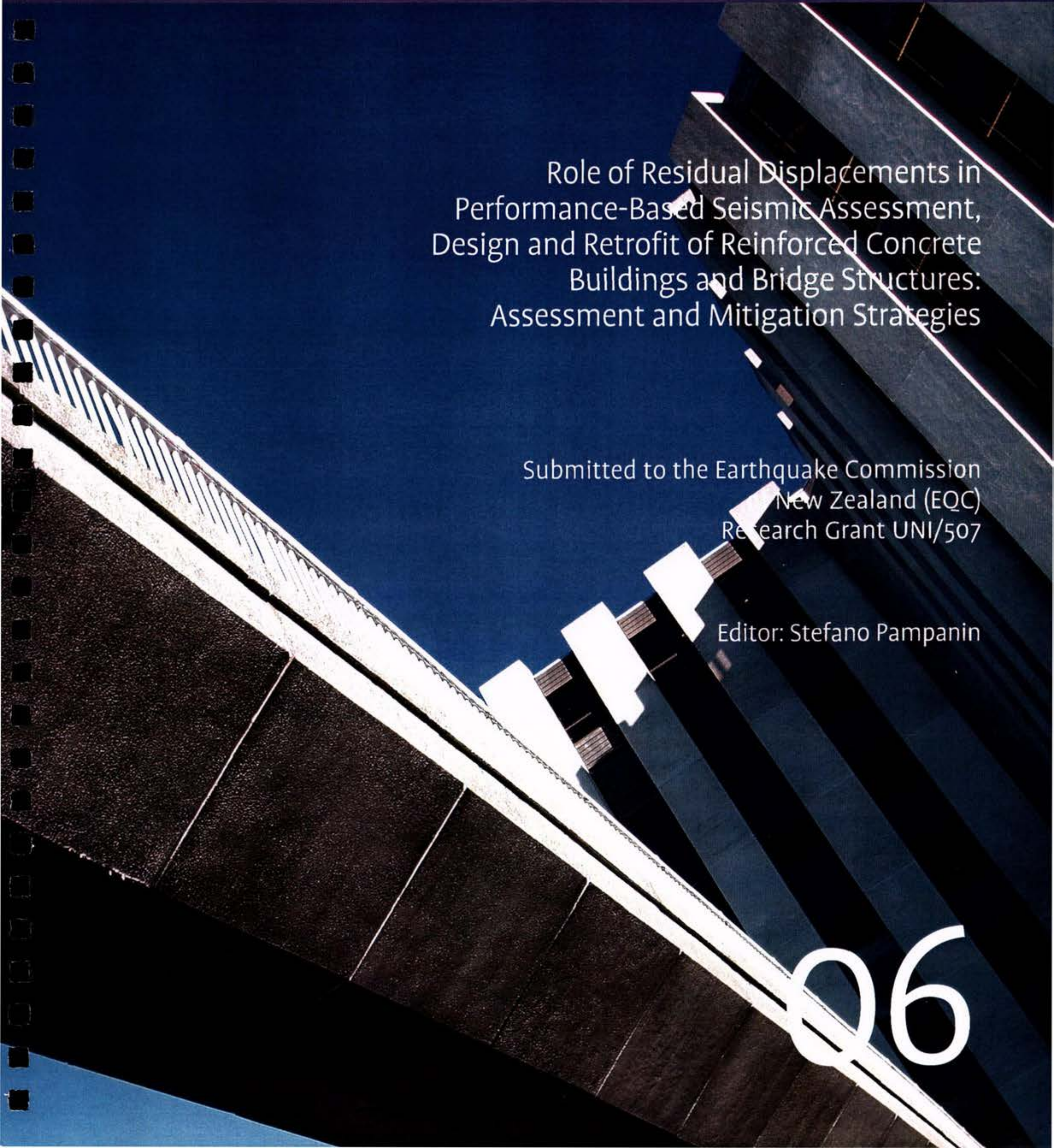
Final Report



Role of Residual Displacements in Performance-Based Seismic Assessment, Design and Retrofit of Reinforced Concrete Buildings and Bridge Structures: Assessment and Mitigation Strategies

Submitted to the Earthquake Commission
New Zealand (EQC)
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Editor: Stefano Pampanin

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FOREWORD

This special publication provides an extended summary of the results obtained as part of the EQC-funded research project "Role of Residual Displacement in Performance-Based Seismic Assessment, Design and Retrofit of Reinforced Concrete Buildings and Bridge Structures: Assessment and Mitigation Strategies" (Grant UNI/507, Oct 2004-Dec 2006).

After an overview of the scope of the project, the research motivations and a summary of the main research outcomes achieved in the last two years, a selection of peer reviewed papers representing direct and tangible outcomes of this project will be herein given.

A special recognition goes to all the members of the project *research team* for their invaluable contribution and unique commitment well beyond the highest expectations:

Associate Investigators: Dr. Constantin Christopoulos (University of Toronto) and Dr. Alessandro Palermo (Technical University of Milan);

Ph.D. candidates: Didier Pettinga (ROSE School, Pavia, Doctor of Philosophy from December 2006), Alejandro Amaris and Dion Marriott (University of Canterbury)

The assistance of the technical staff of the University of Canterbury Civil Engineering Laboratory, in particular Gary Harvey, John Maley, Stuart Toase, Richard Newton is gratefully acknowledged.

The financial contribution provided by the New Zealand Earthquake Commission (EQC) is greatly appreciated. The opportunity to dedicate focused resources to investigate such an emerging while comprehensive topic after few years of preliminary studies carried out by the project principal and associate investigators has played a critical role.

Substantial co-funding have been also provided by the University of Canterbury, the NZ Foundation of Research Science and Technology (FRST-retrofit project), and the National Science and Engineering Research Council of Canada (NSERC). The contribution from the University of Toronto and the Technical University of Milan in terms of salary recovery and overheads components for the Associate Investigators Dr. Christopoulos and Dr. Palermo is also acknowledged.

Ultimately, the Principal Investigator would like to reserve a special thanks to the financial and stimulating support provided by the (EQC/NZSEE) Ivan Skinner Award 2006, which facilitated, in the second part of the year 2006, the development of the overall research project and the presentation of main outcomes at international conferences.

Christchurch, 6 January 2007

Dr. Stefano Pampanin

Principal Investigator

EXTENDED SUMMARY

Research Background and Motivation

Observation from real earthquake event as well as laboratory testing and numerical studies have demonstrated that most structures designed according to current code provisions might sustain substantial residual (permanent) deformations in the event of a design-level earthquake even if they perform exactly as expected. Residual deformations can result in the partial or total loss of a building if static incipient collapse is reached, if the structure appears unsafe to occupants or if the response of the system to a subsequent earthquake is impaired by the new at rest position of the structure. Furthermore, they can also result in increased cost of repair or replacement of non-structural elements as the new at rest position of the building is altered. These aspects are typically not appropriately reflected in current performance design and assessment approaches.

Priestley (1993) discussed the importance of residual deformations when assessing the performance of structures by emphasizing the difficulty and cost associated with straightening structures after a major earthquake before repairs could be carried out. A number of researchers (Kawashima, 1997, MacRae and Kawashima, 1998; Borzi et al. 2001, Pampanin et al, 2002, 2003; Christopoulos et al. 2003, Mackie and Stojadinovic, 2004; Ruiz-Garcia and Miranda, 2006a),; more detailed and recent literature review can be found in the appended papers) have investigated on residual displacement of equivalent Single Degree of Freedom Systems. Only recently the focus has been given to MDOF (Pampanin et al., 2002, 2003; Ruiz-Garcia and Miranda, 2006b).

In particular, a framework for evaluating the level of damage or performance (for a given intensity of the seismic input) based on a combination of maximum and residual response indices has been recently proposed by the main researchers of this project (Pampanin et al., 2002, 2003; Christopoulos et al., 2003).

A preliminary proposal to modify a direct displacement based design method to explicitly include the effect of residual deformations has been presented by Christopoulos and Pampanin (2004)

Further work, based on extensive analytical investigation and experimental validations, is required in order to develop reliable methods to assess and predict the residual deformation in existing or new design structures under seismic response as well as mitigate or reduce the corresponding damage to acceptable (or negligible levels) through a proper design solution or retrofit strategy.

Recognizing the importance of controlling residual deformations, or completely eliminating them, recent developments in precast concrete moment resisting frames (MRF) or jointed shear walls (Priestley at al., 1999, Pampanin et al., 2006) as well as steel MRFs (Christopoulos et al., 2002b) making use of unbonded high strength tendons, have resulted in structural systems which can undergo inelastic displacements similar to their traditional counterparts, while limiting the damage to the structural system and assuring full re-centering capability (reduced or negligible residual deformation).

The extension of the concept to bridge piers and systems has been recently investigated by a number of researchers in the last decade (Mander and Cheng, 1997, Hewes and Priestley, 1999, Kawashima, 2002; Ikeda et al., 2002; Kwan and Billington 2003; Palermo et al., 2005)

Refinement and development of these new technology can lead to valuable strategies for mitigation of residual deformations for new design and retrofit.

Project Objectives and Development

This section provides a summary of the project main objectives as identified and declared in the original proposal (July 2004). Main developments within each single objective are briefly summarized.

The main long-term plan objective of the overall project was/is to develop a rational performance-based design procedure for design assessment and retrofit able to account for and reduce the impact of damage resulting from residual deformation.
It will be noted that the research project achieved very satisfactory results well in line with (when not beyond) the initial, already ambitious, scope and expectations also considering the relatively limited time-frame. It is however important to underline that, given the complexity of the topic as well as the several sub-tasks involved, the investigations have highlighted and partially addressed several issues which require further refinements and continuous and comprehensive investigations in the next future.
It is strong opinion of the authors that such a proposed framework can in fact represent a major breakthrough in earthquake engineering, since the evaluation of damage or collapse level has been typically so far associated with the maximum response (in terms displacements, deformations, drift) occurring during an earthquake, neglecting a fundamental and complementary component of the structural damage.

In view of defining a proper platform for the next generation of code provisions, **three main tasks** were considered in principle as specific objectives of a more general multi-year research project.
Clearly each task described above of them would itself represent a major research program to be carried out and developed in the next few years. The results herein presented, achieved within the direct co-funding of EQC under the grant UNI/507, have provided substantial advancements in all Tasks. Further work is on going (e.g. experimental testing on the shake table to correlate damage and residuals parameter under going) and will be completed in the near future.

- **Task 1) Assessement:** evaluation of residual deformation in existing and newly designed structures. Either analytical and experimental investigation on subassemblies component (beam-column joints, column-to-foundation or pier-to-foundation connections) as well as on entire frame or bridge systems have been performed. Particular emphasis was given to torsion mechanisms due to structural irregularity and second order (P-Δ) effects.

- **Task 2) Mitigation** strategies based on alternative design philosophy. Development of mitigation strategies for traditional solutions based on cast-in-situ concrete. This would include the change of post-yielding stiffness through modification of reinforcement layout, section dimension, geometry of the system as well as the use of high performance material. Either numerical and experimental investigations, based on a shake table test on a prototype one-storey building (consisting of replaceable hinges) and plan irregularity, have been carried out to demonstrate the efficiency of simplified design approach in reducing the expected residual deformation.

- **Task 3)** Development and refinement of new technological solution to reduce the residual deformation using **self-centering systems** (based on rocking or hybrid systems, either using traditional or advanced dissipative systems and materials). The conceptual solutions could be used for either the new design of new structures or the retrofit/upgrading of existing ones. Quasi-static and pseudo-dynamic experimental tests on re-centering beam-column connections as well as column-to-foundation (or bridge piers) following the PRESSS Program concept (hybrid systems) were carried out. In addition, the effects of bi-directional loading regime have been experimentally investigated.

Development and refinement of simplified **modelling** techniques of hybrid (post-tensioning dissipative) systems have also been part of the theoretical investigations. Comparison between the efficiency of alternative modelling methods, either based on lumped plasticity approach or on multi-spring models have been carried out based on the experimental results.

Summary of Research Outcomes

A brief summary of the specific research outcomes, corresponding to the aforementioned project objectives/task is given in this section. More detailed information and results can be found within the selected peer reviewed scientific contributions (and associated references) reprinted for the scope of this internal report.

TASK 1 - ASSESSEMENT

Effect of irregularity on residual displacements: torsion

The performance-based (displacement-based) design framework for residual deformations, previously developed by Pampanin et al. (2002, 2003) and Christopoulos and Pampanin (2004) for 2D regular structures (Fig. 1), has been further extended to the behaviour of 3D irregular (asymmetric in-plan) buildings (Fig. 2, Pettinga, 2006a).

The effects of in plan irregularities, leading to inelastic torsional behaviour was numerically and experimentally investigated.

The seismic response of a set of single-storey systems, comprising of seismic resisting frames, and modelled to represent alternative materials (concrete or steel), was investigated under uni-directional earthquake loading excitations. Different layouts in plan, leading to either torsionally unrestrained or restrained systems, were considered.

The influence of varying torsional restraint was investigated to define how residual diaphragm rotations and centre-of-mass displacements are affected by changing levels of stiffness and strength, or mass eccentricity.
From this investigation a series of alterations and additions to a previously proposed estimation approach and equation were made, such that the SDOF residual drift is converted to that at the building centre-of-mass, and then further extrapolated to the required points of interest within the building plan. The procedure is equally applicable to force-based or displacement-based design approaches (Fig. 3). Using inelastic time-

history results simple demonstrations were made to show that the proposed general equation form and terms within are appropriate, and that with physically meaningful calibrations would be able to reasonably reproduce the observed permanent displacement trends for design purposes.

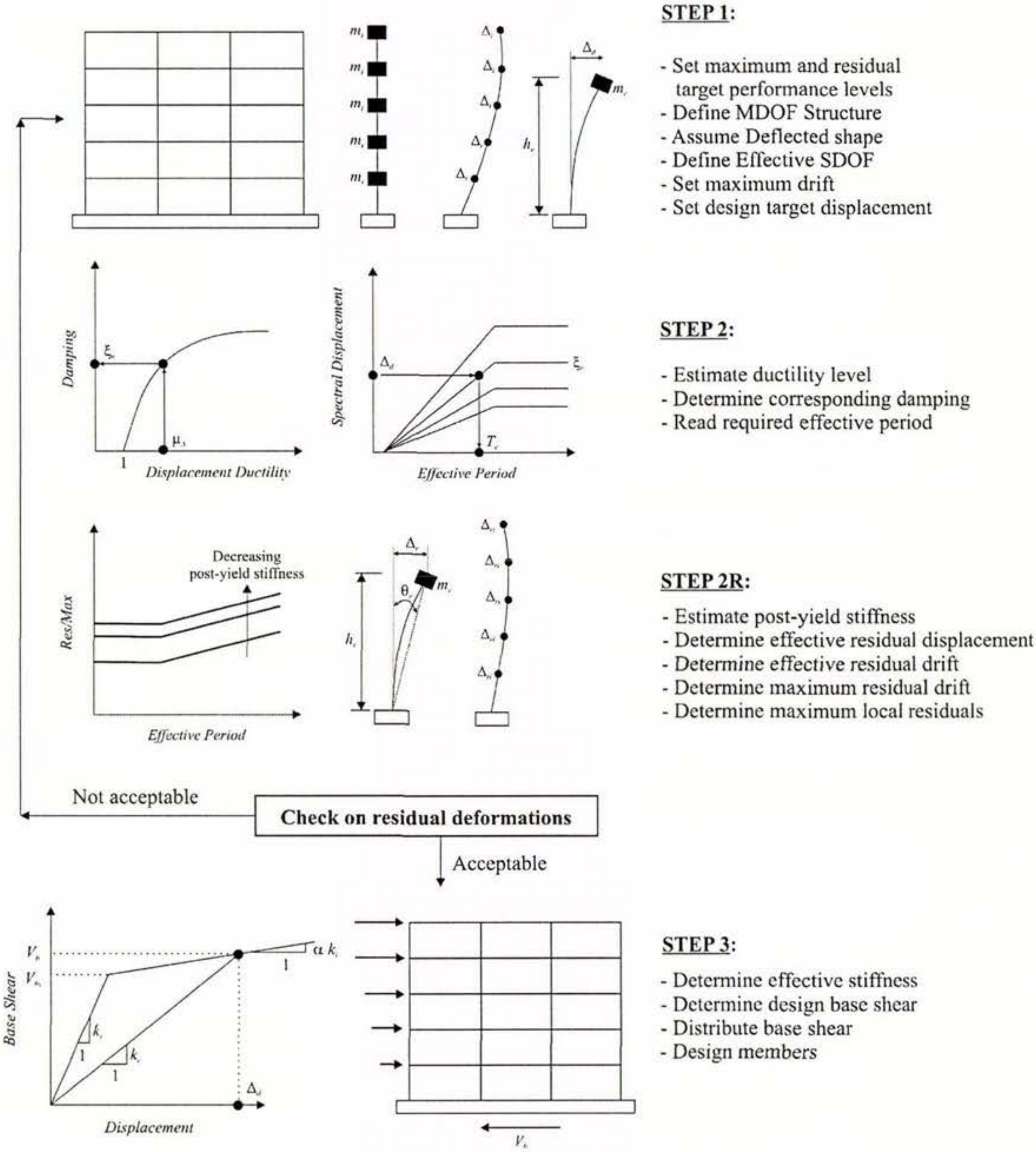


Figure 1. Flowchart of DDBD procedure with explicit consideration of residual deformations (from Christopoulos and Pampanin [2004])

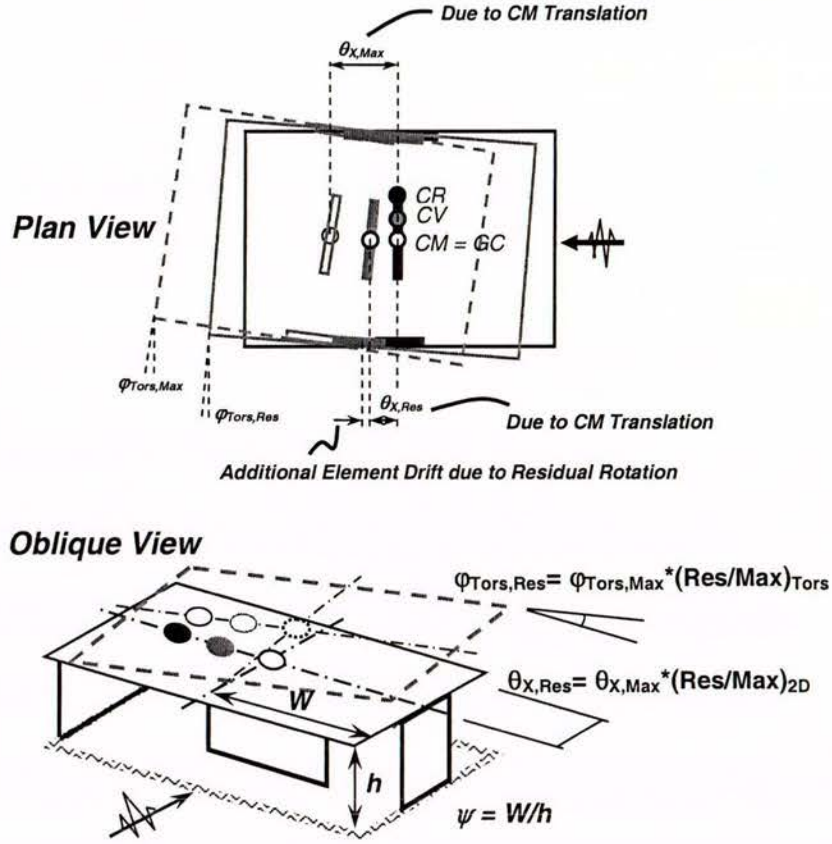


Figure 2. Irregular buildings and torsional response (from Pettinga et al., 2007)

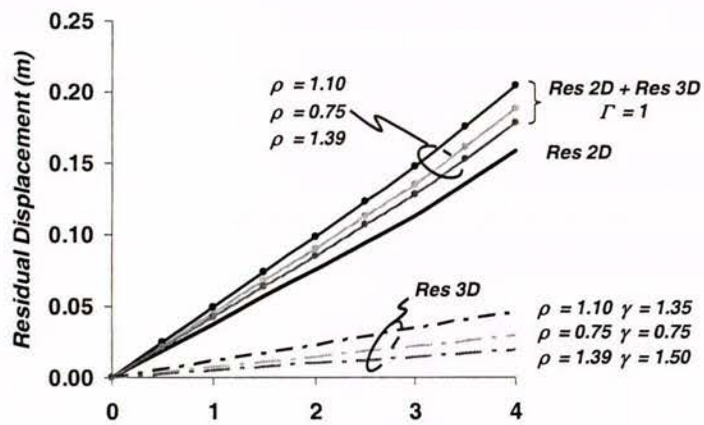


Figure 3. Concept of Total Residual Displacement spectrum. Effects of torsional response to residuals. Indicated are the relatively contributions from translational response at the centre-of-mass (*Res 2D*) and additional perimeter displacements due to residual diaphragm rotations (*Res 3D*) for varying levels of torsional restraint factor ρ (Pettinga et al., 2007)

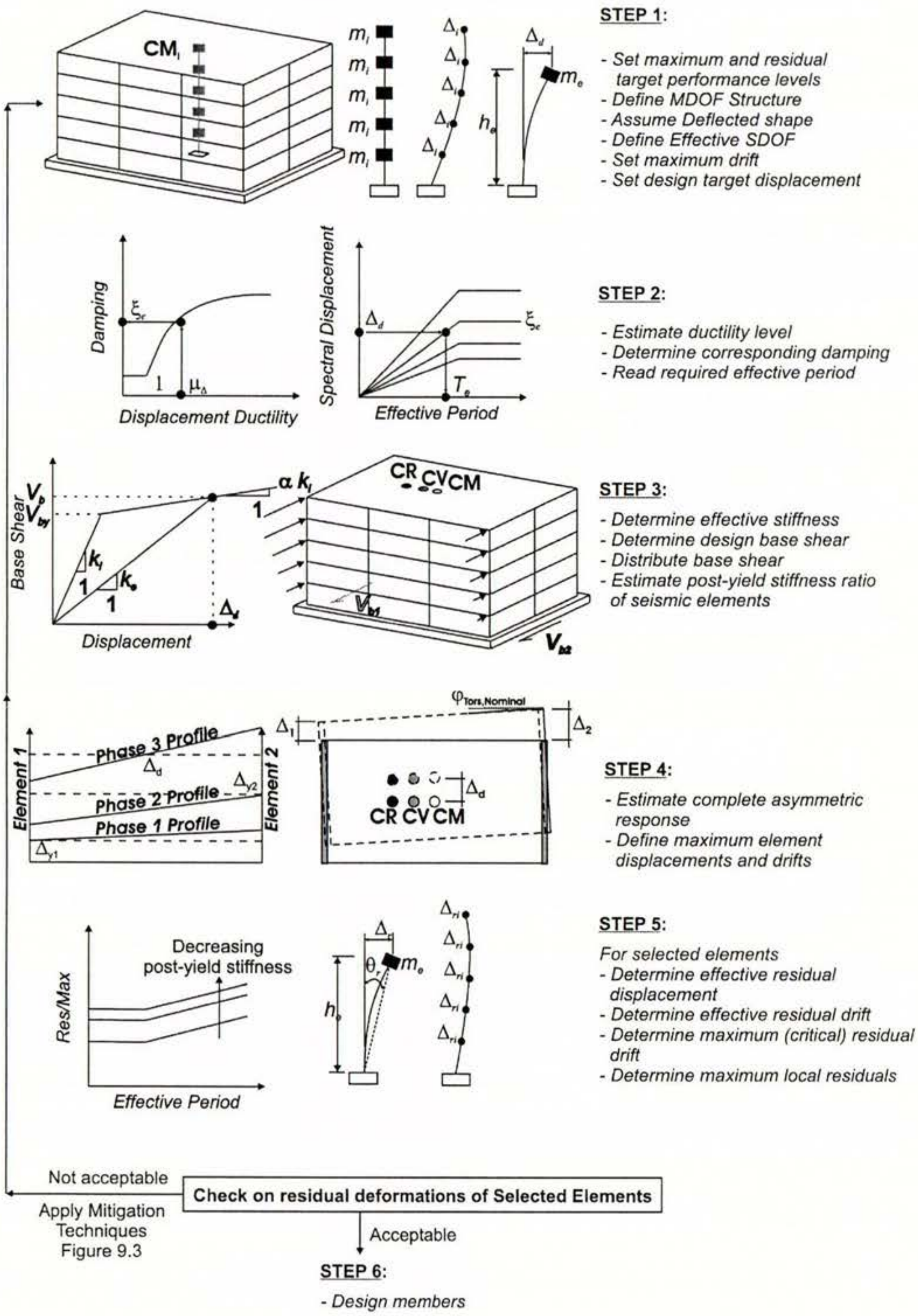

STEP 1:
- Set maximum and residual target performance levels
- Define MDOF Structure
- Assume Deflected shape
- Define Effective SDOF
- Set maximum drift
- Set design target displacement
STEP 2:
- Estimate ductility level
- Determine corresponding damping
- Read required effective period
Damping
Displacement Ductility
Spectral Displacement
Effective Period
STEP 3:
- Determine effective stiffness
- Determine design base shear
- Distribute base shear
- Estimate post-yield stiffness ratio of seismic elements
Base Shear
Displacement
CR CV CM
STEP 4:
- Estimate complete asymmetric response
- Define maximum element displacements and drifts
Element 1
Element 2
Phase 3 Profile
Phase 2 Profile
Phase 1 Profile
CR CV CM
STEP 5:
For selected elements
- Determine effective residual displacement
- Determine effective residual drift
- Determine maximum (critical) residual drift
- Determine maximum local residuals
Decreasing post-yield stiffness
Res/Max
Effective Period
Not acceptable
Apply Mitigation Techniques Figure 9.3
Check on residual deformations of Selected Elements
Acceptable
STEP 6:
- Design members

Figure 4. Revised flowchart of DDBD procedure with explicit consideration of residual deformation based on prediction of complete asymmetric system response (from Pettinga, 2006a).

Second order P-Δ effects

A simplified procedure to explicit include second order effects due to P-Δ action in a Direct Displacement-Based Design method has been proposed (Fig. 5, Pettinga, 2006b). The differences in sensitivity to P-Δ of SDOF elasto-plastic (approximating steel response) and stiffness degrading (reinforced concrete) hysteresis were discussed, from which a proposed multiplicative factor was derived to account for the enhanced performance of reinforced concrete structures.

Parametric investigations, using a suite of seven spectrum-compatible 'massaged' real records, were carried out to assess the influence of differing levels of P-Δ significance, ductility demand and post-yield stiffness ratio. It was found that for most systems considered the proposed design approach can be very effective. Where necessary limits in application and effect were therefore presented based on these results. To further demonstrate the effect of the design approach, a four-storey frame designed for both reinforced concrete and steel response was numerically tested. It was found that the proposed procedure accounting for P-Δ satisfactorily reduces the storey drift amplifications, such that the design performance targets are maintained at the original level even without the need to include second-order effects in the analyses.

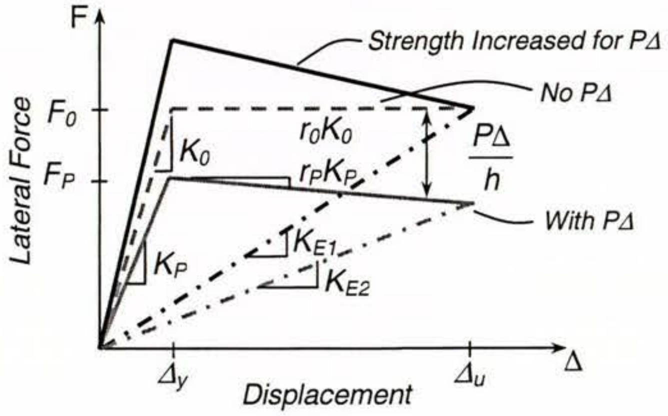


Figure 5. Conceptual Approach to Including P-Δ in DDBD: Force-displacement response showing targeted effective stiffness (Pettinga, 2006b)

Probabilistic formulation of performance-based assessment including residuals

The previous presented tasks and contributions have been focus on a deterministic approach in the evaluation and design to mitigate residual/permanent deformation. As part of the overall project, though not directly funded by this grant, a probabilistic framework for a performance based seismic assessment of structures considering residual deformations has been developed and proposed. First, a probabilistic formulation of a combined 3-dimensional performance matrix, where maximum and residual deformations are combined to determine the overall performance at various seismic intensity levels was presented as an evolution of the performance matrix concept proposed by Pampanin et al., 2002 (Figs. 6-7).

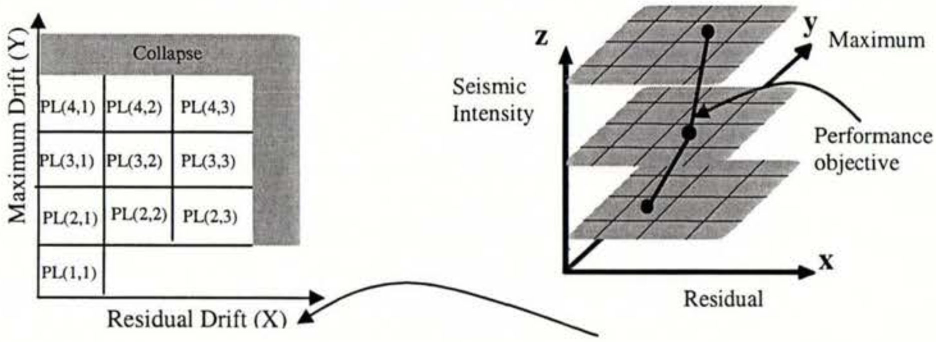
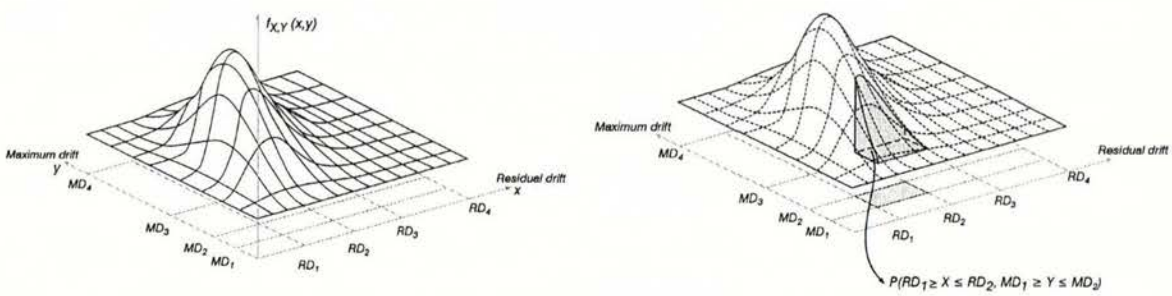


Figure 6. Performance domain considering both maximum and residual deformation indices and 3-Dimensional performance objective matrix (Pampanin et al., 2002).



(a) (b)

Figure 7. Visualization of a Joint probability density function over a performance domain (maximum displacement/drift MD vs. residual displacement/drift MD) from Uma et al., (2006)

Combined fragility curves expressing the probability of exceedence of performance levels defined by pairs of maximum-residual deformations were then derived using bivariate probability distributions, due to the statistical dependence of the two demand parameters.

The significance of evaluating and accounting for residual deformations within a performance based seismic engineering (PBSE) approach was further confirmed via numerical examples on the response of SDOF systems under a selected suite of earthquake records. Fragility curves corresponding to various performance levels, defined as a combination of maximum and residual response parameters, were derived while investigating the effects of hysteretic systems and strength ratios.

Furthermore, the concept of a joined fragility spectrum (based on combined maximum and residual deformation parameters) for design purposes is under development and has been first presented by Uma et al. (2006) along with tentative suggestions for the extension of the proposed methodology to a displacement-based design probabilistic approach with targeted confidence of achieving different performance levels.

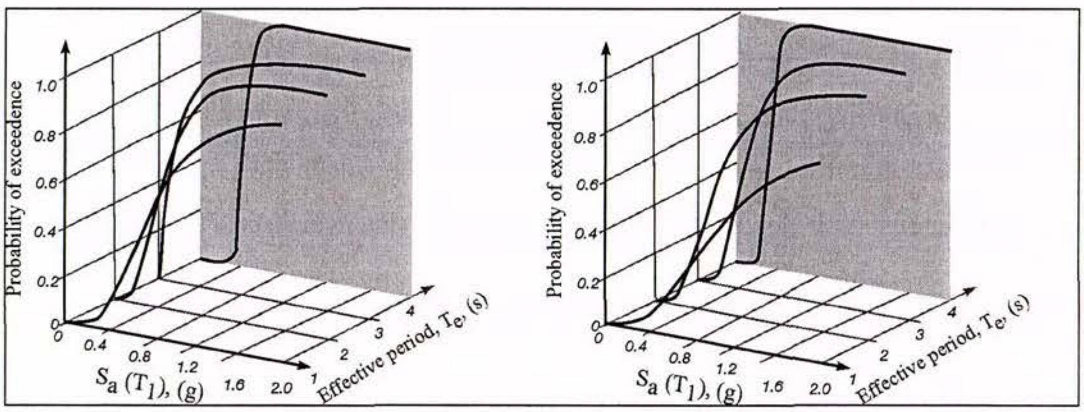


Figure 8. Joined Maximum –Residual constant ductility fragility spectra for assessment and design purposes (Uma et al., 2006)

TASKS 2 & 3 - MITIGATION TECHNIQUES AND DESIGN STRATEGIES

Having estimated the expected residual deformations in a structure, engineers are faced with the problem of reducing them to meet the targeted performance levels under predefined seismic hazard levels. Alternative approaches have been herein investigated, based on either numerical and analytical investigation, either relying upon:

- a) the use of simplified methods based on traditional technology – (Task 2)

  or
- b) the use of post-tensioning techniques to provide self-centering (as in the jointed ductile connections or hybrid systems, developed under the U.S.-PRESSS Program, Priestley, 1991, Priestley et al, 1999) – (Task 3)

TASK 2 – Alternative mitigation strategies based on traditional techniques (not relying on post-tensioning)

Previous studies have identified the post-yield stiffness as a primary factor influencing the magnitude of residual deformations in SDOF and MDOF structures. In this part of the project, a series of simple approaches to increase the post-yield stiffness of traditional framed and braced systems for the purpose of reducing residual deformations are investigated. These methods do not utilize re-centering post-tensioned technology.

The feasibility of altering the lateral post-yield stiffness of structural systems by i) using different reinforcement materials with beneficial features in their stress strain behaviour (Fig. 9) ii) re-designing the section geometry, reinforcement layout and properties of primary seismic resisting elements, and iii) introducing a secondary elastic frame to act in parallel with the primary system (Fig. 10), was numerically investigated first. The efficiency of each of these techniques has been investigated through monotonic and cyclic moment-curvature and non-linear time-history analyses (Fig. 11). Of these approaches the design and introduction of an elastic secondary system was found to be most effective and consistent in reducing residual deformations. A simplified design approach for achieving the desired increase of a system's post-yield stiffness has been also presented.

Experimental validation on the efficiency of these simplified methods in reducing the residual deformation in irregular structures prone to inelastic torsional response, have been carried out via shake table tests in the Structural Laboratory of the Univeristy of Cantebury (Pettinga, 2006, Chapter 7, Figs. 13-14). The effects of implementing the proposed mitigating techniques were assessed by comparing the response of a benchmark specimen (Castillo, 2004) as shown in Table 1 (Pettinga, 2006)

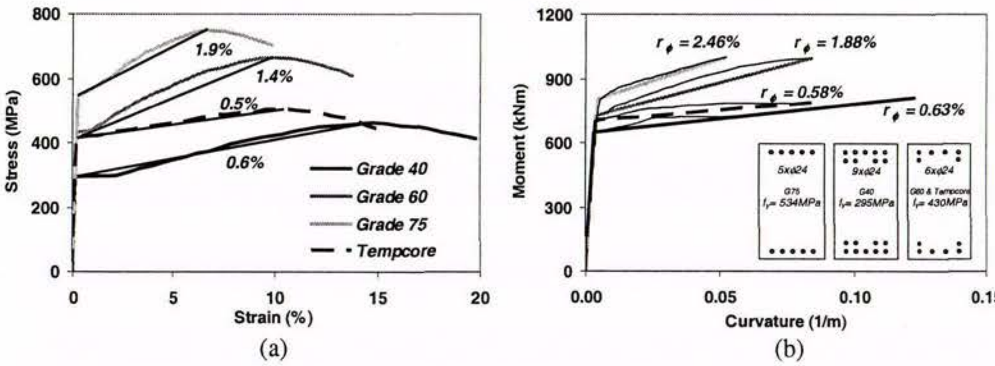


Figure 9. Reinforcing steel comparisons (a) steel stress-strain curves and bilinear strain-hardening projection with strain-hardening ratio values shown (b) bi-linearized M-ϕ plots for sections of approximately equal strength with differing reinforcing strengths and strain-hardening

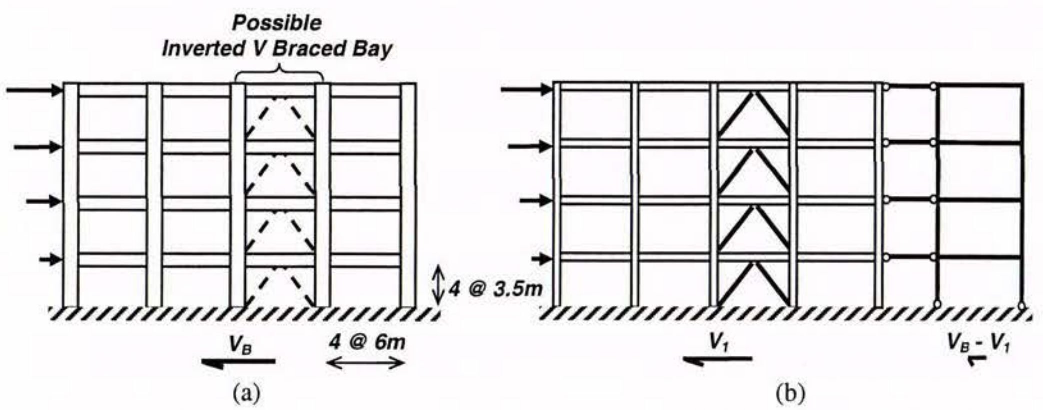


Figure 10 (a) elevation of primary system RC, steel MRF, or BRBF (b) schematic representation of primary BRBF rigidly connected to secondary internal elastic MRF.

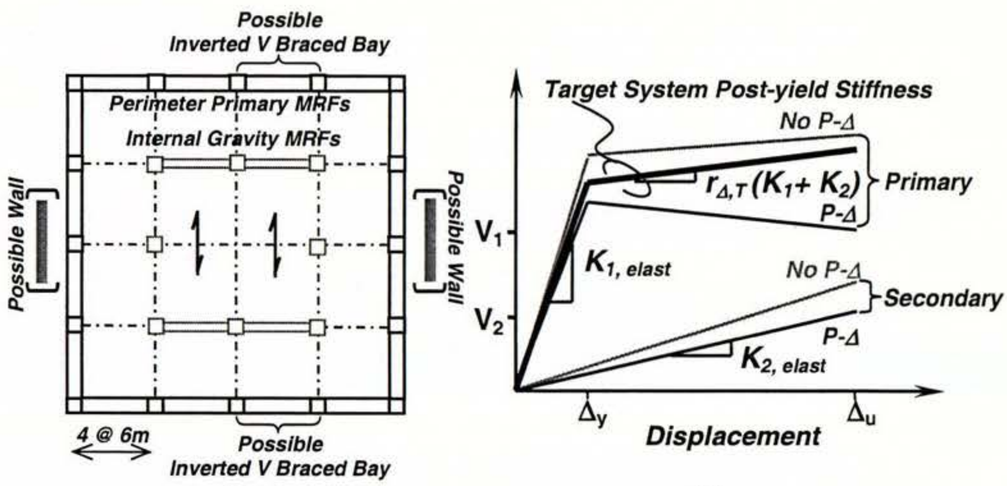


Figure 11. (a) Plan of study building showing primary moment-resisting frames (MRF) and internal secondary gravity MRF positions (also shown are possible structural wall or Inverted-V buckling-restrained braced frame (BRBF) positions) (b) force-displacement representation of individual primary and secondary resisting frames and total target system response

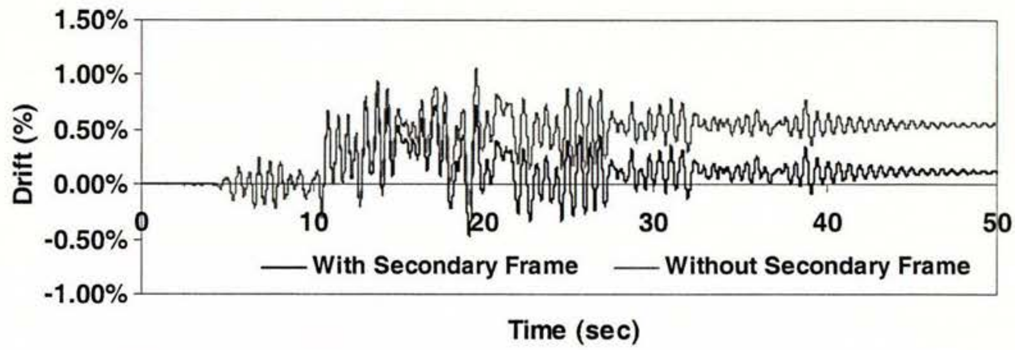


Figure 12. Time-history response at the Center-of-Force (CoF) of the initial 'Bare' BRBF with no secondary frame and comparison with the response when a secondary frame is used.

Table 1. Summary of maximum and residual drifts for experimental results of Castillo and Pettinga comparing the effect of changing the system strength and plastic-hinge post-yield stiffness (note that residual drifts from Castillo are estimated as they were not recorded)

		Model 5-1		Model 5-3A		Model 5-3B	
		θ_{max} (%)	θ_{res} (%)	θ_{max} (%)	θ_{res} (%)	θ_{max} (%)	θ_{res} (%)
Castillo	**Frame 1**	3.46	~2.1	3.95	~2.3	4.32	~2.5
	Frame 2	3.46	~2.1	4.63	~3.0	5.34	~3.9
Pettinga	**Frame 1**	2.96	0.88	3.17	1.01	2.45	0.54
	Frame 2	2.68	0.66	3.13	0.85	2.75	0.39

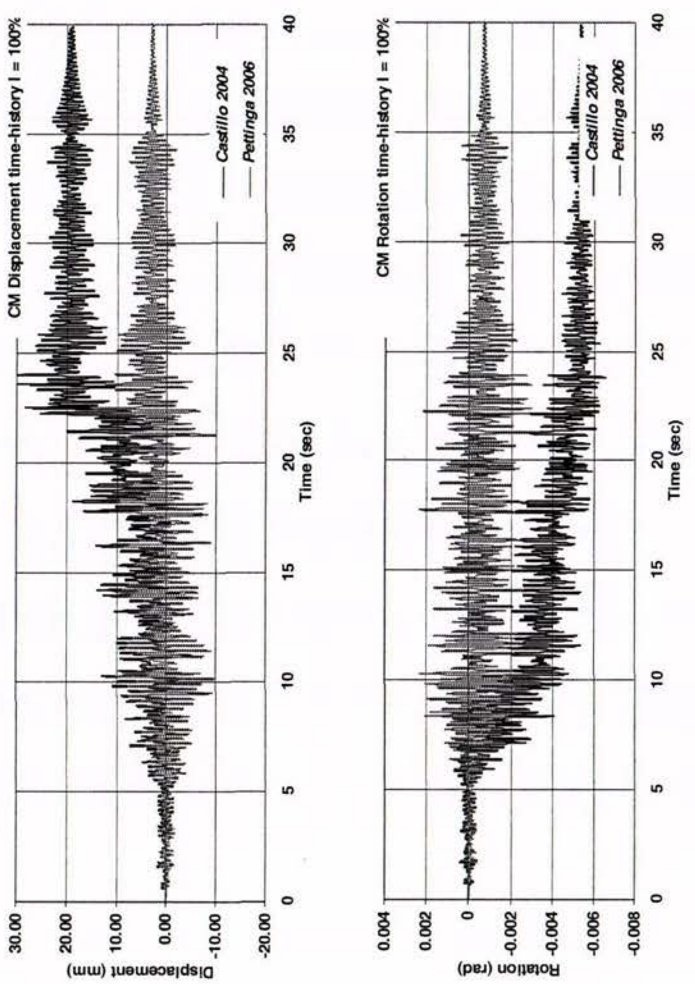


Figure 13. Model 5-3B (e_{rx} = -0.092 e_{vx} = -0.052) CM displacement and rotation response comparison as tested by Castillo (2004) and Pettinga (2006).

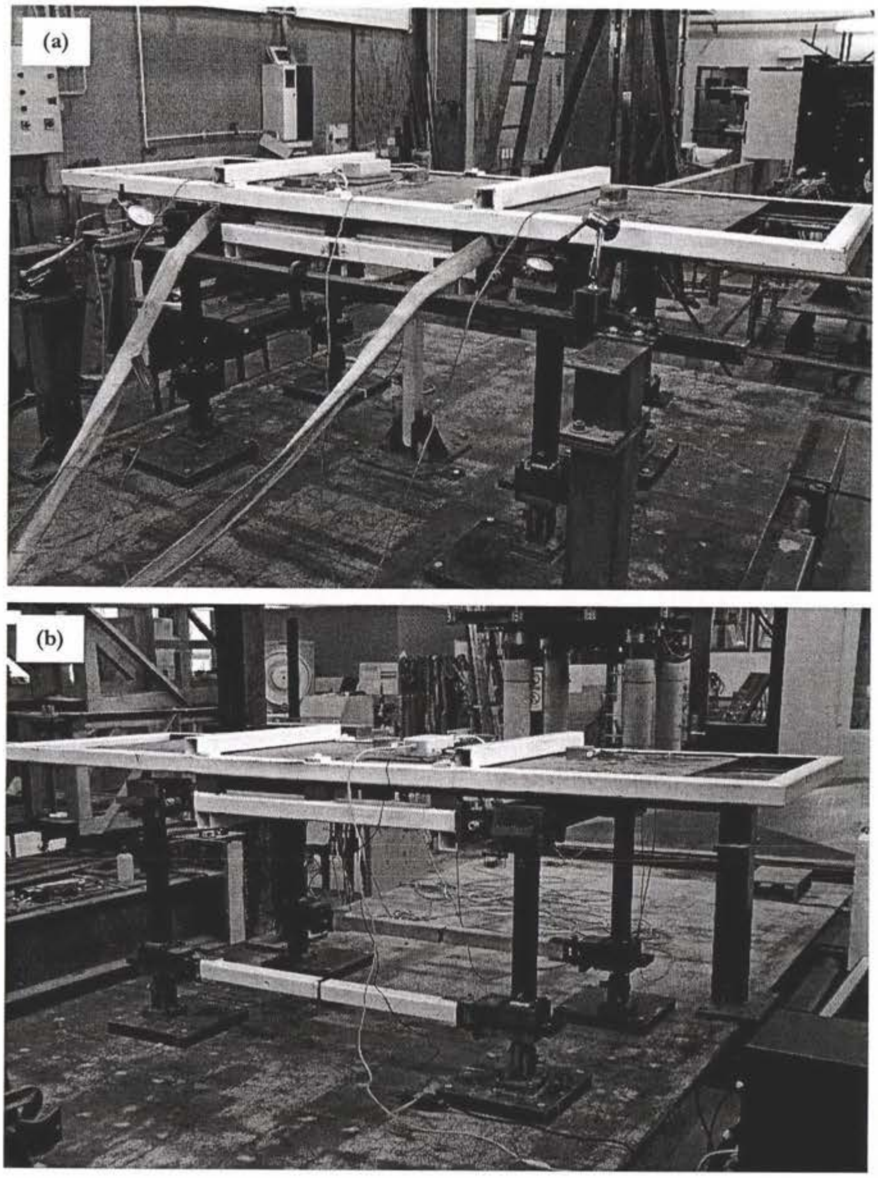


Figure 14. Experimental prototype for evaluation of effects of residual due to torsional response and development of mitigation strategy:
(a) Torsionally unrestrained model (b) Torsionally restrained model

TASK 3- Implementation of self-centering systems: buildings

Recent developments on high performance seismic resisting precast concrete frame systems, based on the use of unbonded post-tensioned tendons with self-centring capabilities in combination, when required, with additional sources of energy dissipation, have been presented in (Pampanin et al., 2006). Alternative arrangements for jointed ductile connections to accommodate different structural or architectural needs have been implemented and validated through quasi-static cyclic tests on a series of exterior beam-column subassemblies under uni- or bi-directional loading regime. The results confirmed the unique flexibility and efficiency of these systems for the development of the next generation of seismic resisting structures, able to undergo high inelastic displacement with limited level of damage and negligible residual displacement when compared to traditional monolithic (cast-in-situ) ductile solutions.

In order to further emphasize the enhanced performance of these systems, a comparison with the experimental response and observed damage of 2-D and 3-D monolithic beam-column benchmark specimens designed according to the NZ3101:1995 seismic code provisions was carried out (Figs. 15-19). The reliability and simplicity of recently implemented special code provisions for the design and analysis of jointed ductile systems was also confirmed by satisfactory results of analytical-experimental comparison. In addition, the practical feasibility and efficiency of simple technical solutions to connect precast floor systems and lateral resisting frame systems, without incurring in damage due to displacement incompatibilities were experimentally demonstrated (Figs. 18-19). The reliability of recently implemented special code provisions for the design and analysis of jointed ductile hybrid systems was also confirmed.

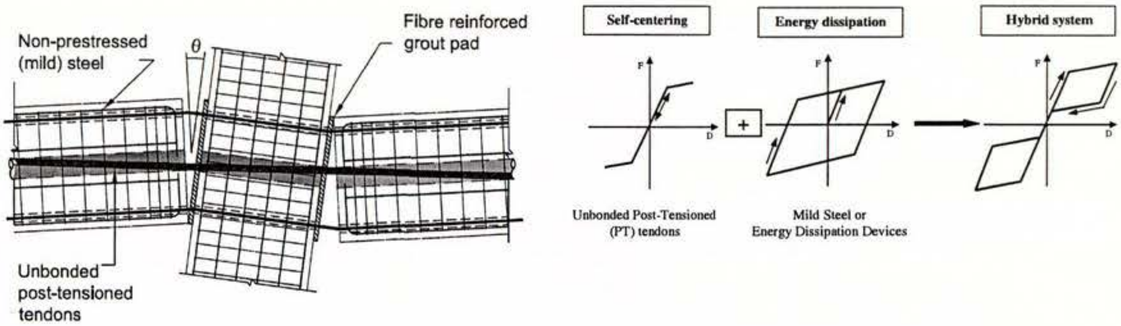


Figure 15. Mechanism of hybrid (post-tensioned and dissipative) PRESSS-type beam column joints and idealized flag-shape hysteretic behaviour (NZS3101:2006,Appendix B; fib, 2004).

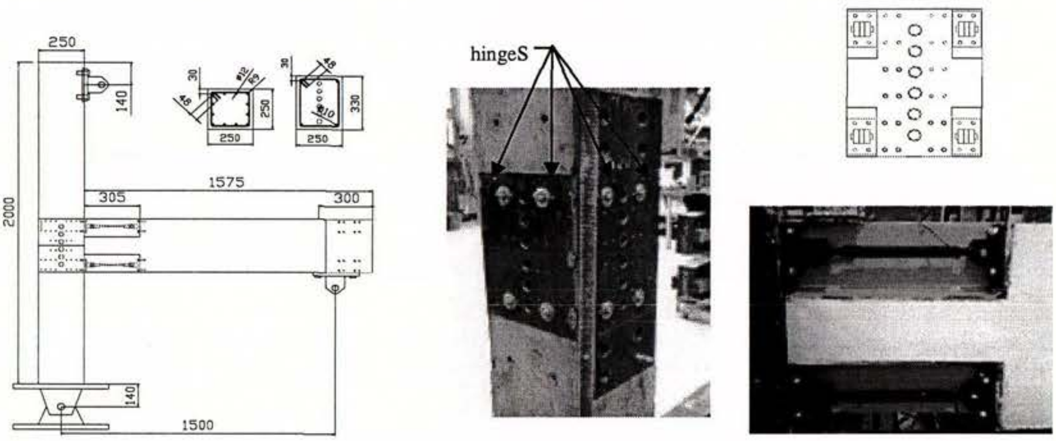


Figure 16. 3-D modular configuration of the Hybrid beam-column joint.
Location of "double hinge" shear keys and replaceable energy dissipaters.

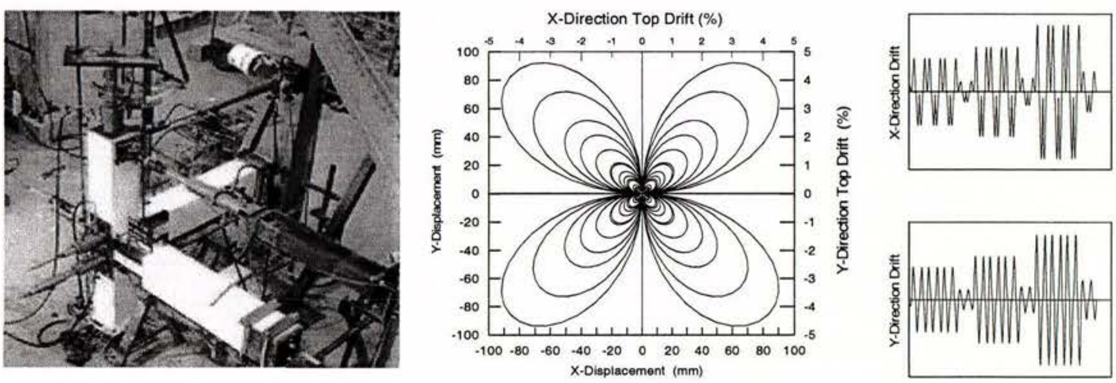


Figure 17. Test set-up and "four clove" bi-directional displacement regimes.

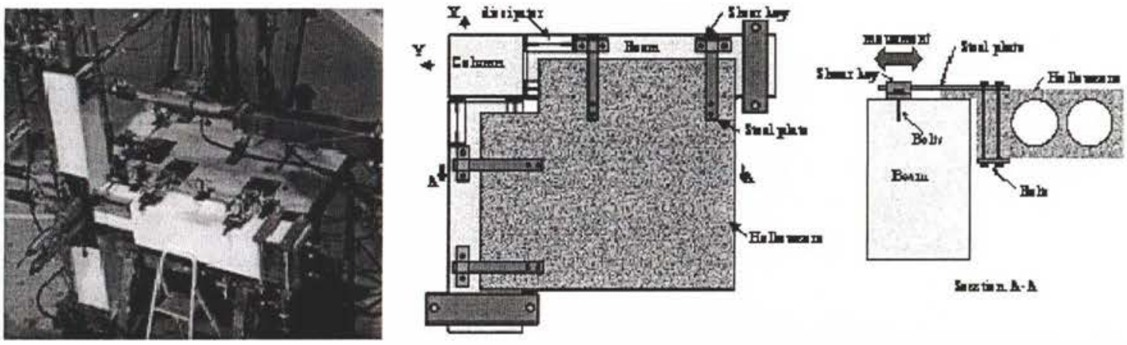


Figure 18. 3-D beam-column joint with articulated floor unit. Overall view, concept and connection details.

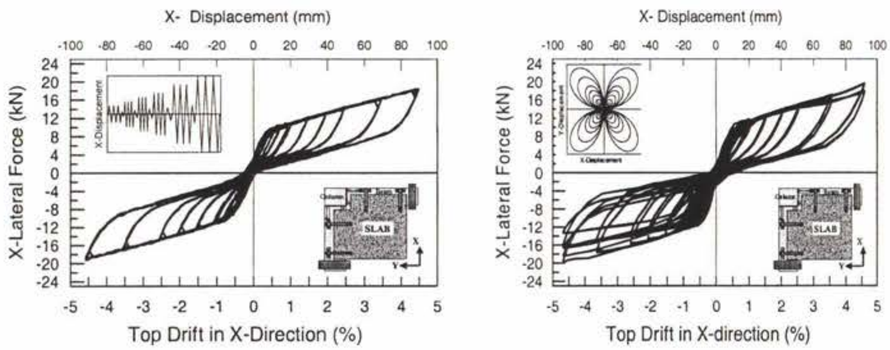


Figure 19. 3-D beam-column joint with articulated (jointed) floor solution. Response in X-direction due to uni- directional and bi-directional testing regime.

Implementation of self-centering systems: bridges

The extension and application of similar technology and seismic design methodologies to bridge piers and systems has been recently proposed in literature as a viable and promising alternative to traditional monolithic or precast construction. In this research project, further confirmations of the unique design flexibility, the ease of construction and the high seismic performance of jointed ductile hybrid systems, combining re-centering and dissipation capabilities, have been investigated (Fig. 20). Simple design methodologies and modeling aspects, able to fully control the seismic response of these systems, have been developed, based on minor modification to the theory presented in the NZ Concrete Standard Code NZS3101:2006.

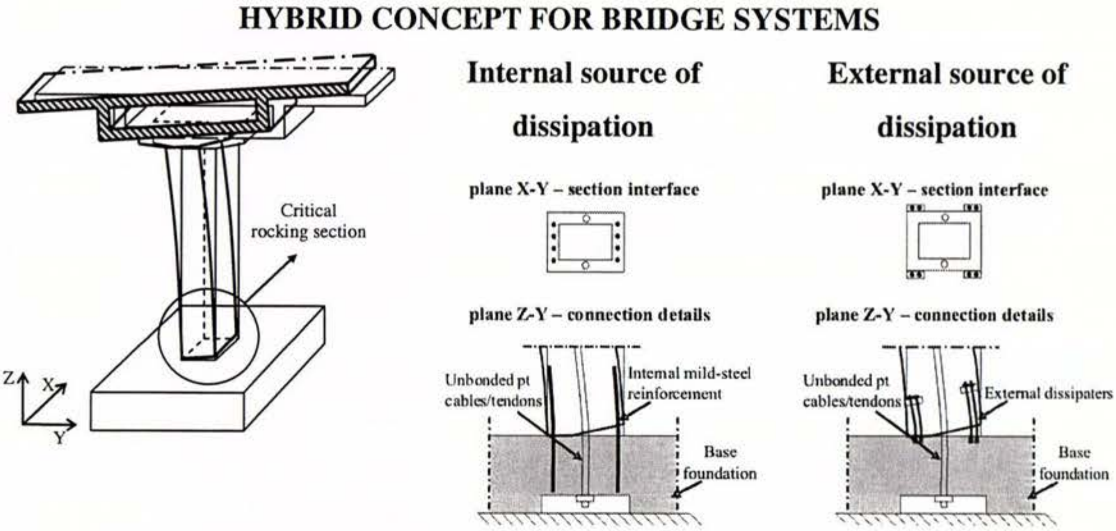


Figure 20 Hybrid bridge pier with internal or external dissipation source.

A series of quasi-static cyclic and pseudo-dynamic experimental tests under uni- or bi-directional loading regime hace been carried out on alternative hybrid configurations (Figs. 21-23).

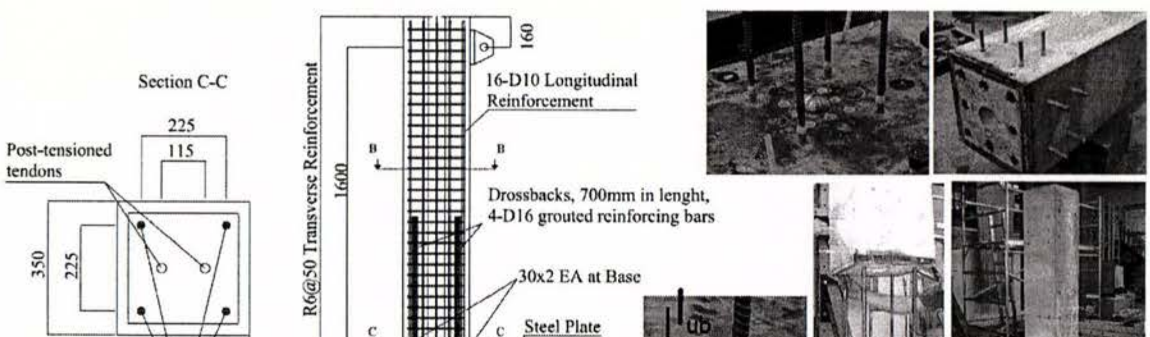


Figure 21. Hybrid specimen with post-tensioning and internal dissipaters, reinforcement layout, geometry and construction details (specimen PT1 adopts same number and configuration of post-tensioned tendons)

Variations of the ratio between the post-tensioning steel and the mild steel, level of initial prestress and type of dissipaters (internal or external replaceable) were experimentally investigated. Lower level of damage and negligible residual/permanent deformations were observed in all the hybrid solutions when compared to the experimental response of the benchmark specimens, representing a typical monolithic (cast-in-situ) ductile solution. In addition, valuable confirmations of the efficiency of the simplified analytical procedure adopted in the design and modeling were obtained (Fig. 23).

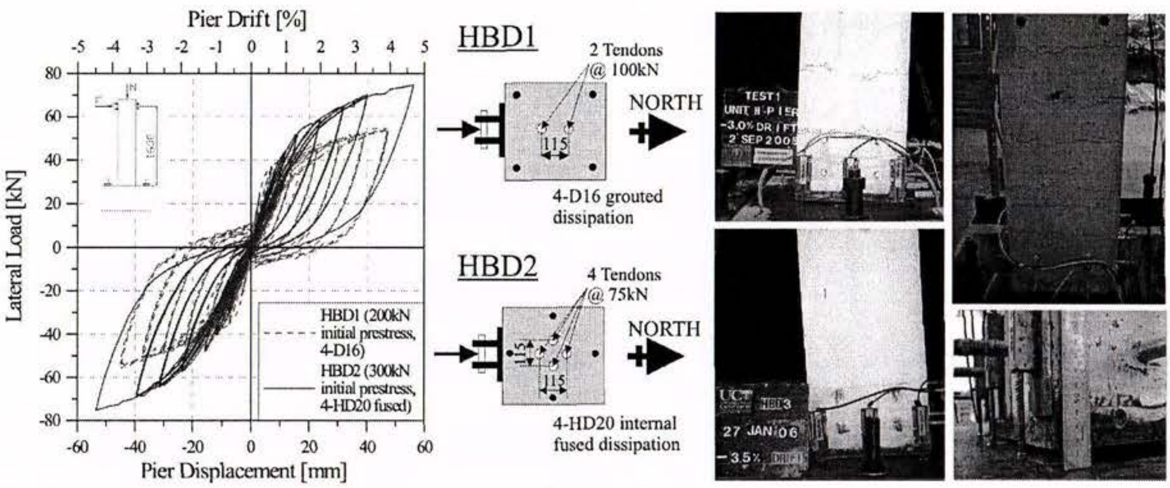


Figure 22. Experimental results of hybrid specimen a) Force vs. displacement response, b) Photos indicating damage at 3.0% and 3.5% drift limit respectively.

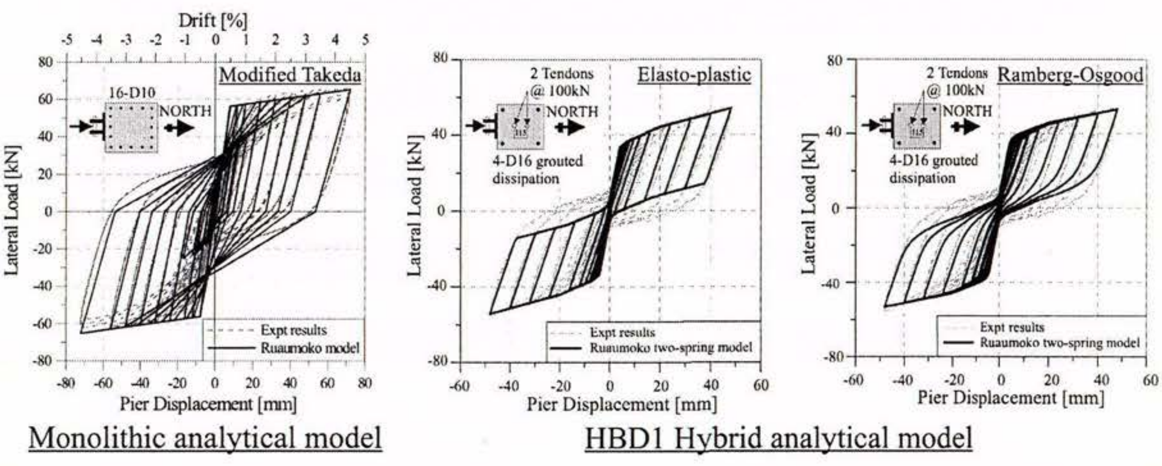


Figure 23. Validation of the analytical model (real predictions) with the experimental results: a) Monolithic specimen (MON1) using a single rotational spring with a Modified Takeda rule ; b) Hybrid specimen (HBD1) using two rotational springs in parallel: a Non-Linear Elastic rule combined with either an Elasto-plastic or a Ramberg-Osgood rule

Application to Advanced Retrofit Strategies

The recent emphasis given on residual deformation, re-centering capability as well as limited level of damage thanks to a controlled rocking system have resulted in the development and proposal of advanced seismic retrofit strategy and technology able to provide an higher performance with limited level of damage and permanent deflection.
An overview of innovative solutions based on a combination of heritage from the past and new technology has been given in Pampanin (2006).
The significant advantages of hybrid or controlled rocking systems (in terms of limited level of damage, control of the stress level acting as fuse) could for example have in fact suggested the use of a *selective weakening* intervention for either beam-column joints or wall systems (Ireland et al., 2006, Fig. 24).

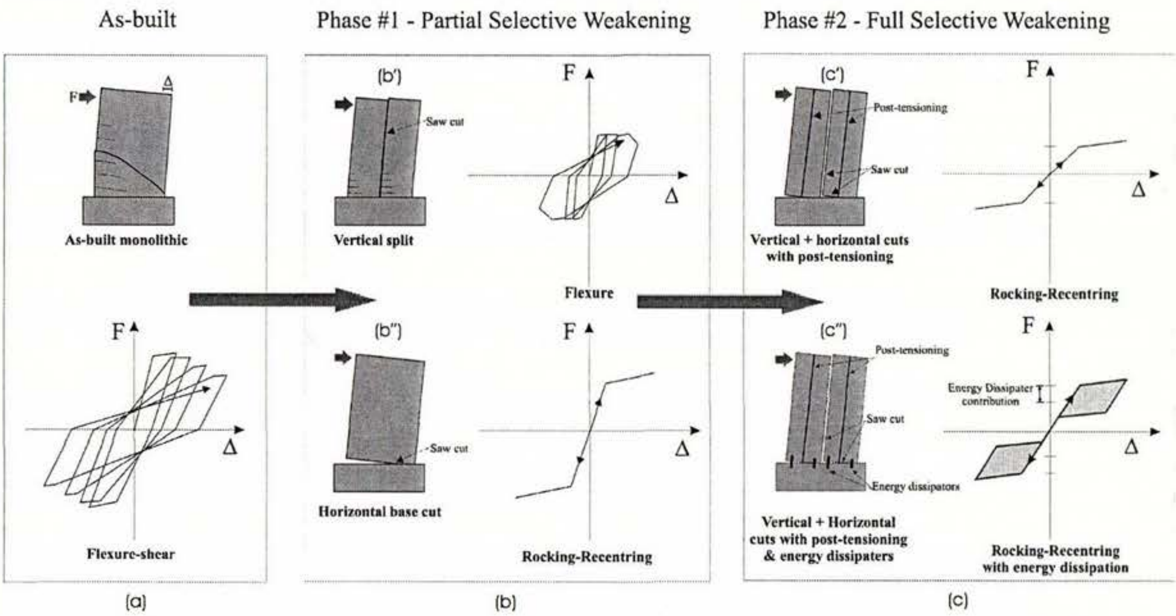


Figure 24: Concept of Selective weakening as a retrofit strategy (Ireland et al., 2006).
Expected damage and hysteresis structural response:
(a) as-built wall; (b) partial selective weakening; (c) full selective weakening

By saw cutting the longitudinal bottom reinforcement of a gravity load dominated beam or of a shear-dominated wall a better control of the overall mechanism can be achieved, according to hierarchy of strength principles A flexure-dominated rocking mechanism can be activated, which is able to guarantee limited level of damage in the structural member as well upper limit level of stress (fuse action) directed to the beam-column joint panel zone or to the existing foundation protecting weak links of the fuse. Moreover, shear walls with low aspect ratio in existing buildings could be suggested to be splitted into two adjacent rocking coupled walls, with significant reduction of shear failure concerns as well as overturning demand to the foundation (Fig. 24, Ireland, 2006). The implementation of the conceptually proposed selective weakening solution existing shear-dominated wall systems has been successfully tested by Ireland (2006).

Further enhancement of this behaviour could also be achieved by using advanced energy dissipation devices (e.g. viscous-elastic, friction, SMA, combined in advanced flag-shaped systems, Marriott, 2006). Shake table tests are currently under going.

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SELECTED PUBLICATIONS

Direct **outcomes** of this EQC-funded research project are listed in the following sections divided in Project Tasks. In **bold** are the papers reprinted in this final report;

* indicate other publications related to the overall research topic but not necessarily directly funded by this grant.

ASSESSEMENT OF RESIDUAL DEFORMATIONS

Effects of irregularity: Torsion

Pettinga, D., Pampanin, S., Christopoulos, C., Priestley, M.J.N.

The Role of Inelastic Torsion in the Determination of Residual Deformations, Journal of Earthquake Engineering, accepted for publication, 2007

Pettinga, D.

Developments in the Prediction and Mitigation of Residual Deformations due to Seismic Demand, including Asymmetric Structural Response", Ph.D. Dissertation, ROSE School, Pavia, Italy, Dec 2006

Pettinga, J.D., Priestley, M.J.N., Pampanin, S., Christopoulos, C.

Accounting for the Effects on Residual Deformations due to Torsional Response, NZSEE Conference, Napier, March 2006 <u>(Awarded Best Student Paper)</u>

Pettinga, D., Pampanin, S., Christopoulos, C., Priestley, M.J.N.

Effects of Irregularities on the Residual Displacements of Structures Subjected to Inelastic Torsional Response", 4^{th} Workshop on Irregular Concrete Structures, Thessaloniki, Aug 2005

Second order P-Δ effects

Pettinga, D.

Accounting For P-Delta Effects In Structures When Using Direct Displacement-Based Design", Individual study, ROSE School, Pavia, Italy, Dec 2006

Probabilistic formulation of performance-based assessment including residuals

Uma, S.R., Pampanin., S., Christopoulos, C.

A Probabilistic Framework to Develop Performance Objectives Based on Maximum and Residual Deformations, 1st ECEES, Geneva, Switzerland, paper n. 731, Sept 2006

**Uma, S.R., Pampanin, S., Christopoulos, C.*,

Probabilistic formulation of a performance-based matrix combining maximum and residual deformations, *Proceedings of the Annual NZSEE Conference*, Napier, March 2006

Accounting for the Effects on Residual Deformations due to Torsional Response

2006 NZSEE Conference

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ABSTRACT: Recent developments in performance-based seismic design and assessment approaches have emphasised the importance of properly assessing and limiting the residual (permanent) deformations typically sustained by a structure after a seismic event, even when designed according to current code provisions.

In this contribution, the performance-based design framework for residual deformations, previously developed by the authors for 2-D regular structures, is further extended to the behaviour of 3-D irregular (asymmetric in-plan) buildings. The seismic response of a set single storey systems, comprising of seismic resisting frames, and made of alternative materials (concrete or steel), is investigated under uni-directional earthquake loading excitations. Different layouts in plan, leading to either torsionally unrestrained or restrained systems, are considered.

Sensitivity analyses are carried out in order to identify the influence of varying levels of torsional restraint on the residual deformations/displacements in the response of a 3-D irregular building, the irregularity being given by an imposed mass eccentricity.

1 INTRODUCTION

As part of the development of performance-based seismic design procedures, it is becoming increasingly recognised that such approaches should take into account the likely residual deformations of structures. A number of researchers (MacRae et al. 1993a & 1993b, Priestley 1993, Kawashima 1997, Borzi et al. 2001, Christopoulos et al. 2003) have investigated the residual deformation behaviour of structures, however to date these studies have been limited to SDOF oscillators or simple 2-D frame systems. Contrary to this, extensive studies on the maximum torsional response of buildings have been carried out, with significant advances made in recent years to develop approaches that act to control twist induced displacement demands for a wide range of building forms. There remains however no explicit mention of residual deformations due to torsional response.

A need therefore remains for a simple but comprehensive design procedure accounting for residual deformations that accounts for second-order effects (i.e. P-Δ) and irregularities both in plan and elevation. Such an approach should be flexible such that it can be applied in both force-based (FBD) and displacement-based design (DBD) contexts, using some form of simplified 2-D approximation to the full 3-D system.

A comprehensive research program has been initiated with the intent to further investigate the role of residual deformations within performance-based design and assessment approaches, extending the conceptual outline presented by the authors (Christopoulos et al. 2003, Pampanin et al. 2002 & 2003) to the 3-D response of irregular buildings. This contribution will focus on the behaviour of simple frame structures; however the results are also representative of similar structural wall systems. Some basic comparisons are drawn between different construction forms with models representative of reinforced concrete and steel materials.

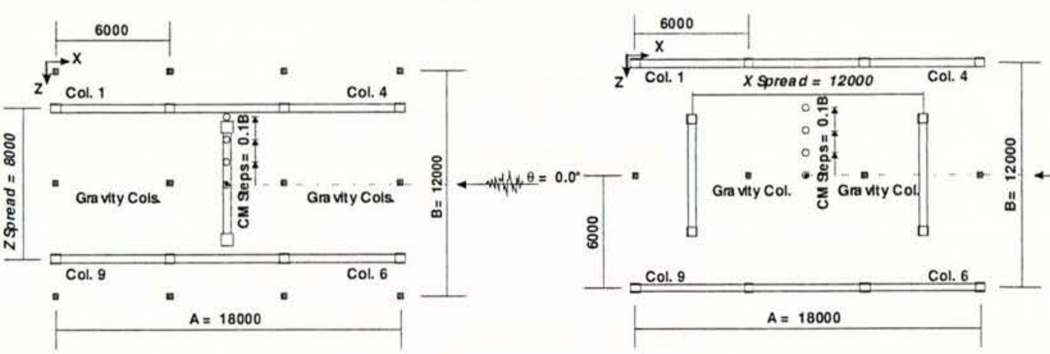
2 INVESTIGATION OUTLINE

A series of single storey 3-D structures consisting of frames or walls have been considered, and their response to earthquake excitation assessed using inelastic time-history analyses. Both torsionally restrained and unrestrained configurations, according to definitions promoted by Paulay (1996 & 2000) and Castillo (2002 & 2004), of seismic resistance are included.

Systems with mass eccentricity or strength and stiffness eccentricity were considered in the study, however only results for the mass eccentric frames are shown in the following sections. A recent study by Peruš & Fajfar (2005) has shown that maximum torsional response due to mass or combined strength and stiffness eccentricity is generally similar; the residual deformation results for both types of eccentricity generally reflect this finding. The findings presented below are primarily related to the response of unrestrained and restrained frame buildings, however the general trends are also applicable for similar wall systems. Further background information and results are given by Pettinga et al. (2005).

3 SYSTEM AND ANALYSIS DEFINITION

Figure 1 shows the plan views for the torsionally (a) unrestrained and (b) restrained frame buildings considered for the restraint studies. Included are the incremental positions of the CM and the applied earthquake angle of attack with respect to the CG. The wall systems considered have the same overall dimensions and layout, with elements sized and reinforced to satisfy NZS3101:1995 code requirements.



(a) (b)

Figure 1. Plan configuration of (a) torsionally unrestrained (b) torsionally restrained frame buildings used to investigate the interaction effects of varying radii of gyration of strength and/or mass.

3.1 Structural Design and Modelling

The structures were designed in each principal direction using a Direct Displacement-Based Design approach (Priestley & Kowalsky 2000, Pettinga & Priestley 2005) with a 2.5% target drift and equivalent viscous damping values typical of reinforced concrete connections ($\xi \approx 20\%$). When comparing the response of alternative (material-wise) structures, the same design strengths and overall monotonic behaviour of the connections were considering, while alternative hysteretic rules, more appropriate for steel or concrete (i.e. Elasto-Plastic, EP or Takeda, TK), were implemented for the inelastic time-history models. A lumped plasticity approach was adopted for the numerical model, implemented in

Ruaumoko 3-D (Carr 2005), consisting of Giberson one-component frame elements for both beams and columns (with an additional moment-axial load interaction yield surface). At this stage of the investigation, for simplicity, no interaction between the flexural capacities in the column element principal directions under bi-axial demand has been considered.

3.2 Time-history Analysis Excitations

The time-history analyses were carried out using a suite of five accelerogram pairs that included four real earthquakes and one artificial pair (created from two random seeds in *SIMQKE*). The real records were scaled to match a modified Eurocode 8 (CEN 2002) pseudo-displacement design spectrum (0.5g PGA; Soil Type B) with an assumed damping of $\xi = 20\%$ (Figure 2). The principal component, selected based on the maximum area under the 20% damped pseudo-displacement spectrum for each direction, was scaled to minimise the average root-mean-square of the observed 20% damped spectrum from the target design displacement spectrum (adapted from Bommer & Acevedo 2004) over the period range zero to four seconds. It is worth noting that the selection of such a wide period range for spectrum compatibility was due to the intent to extend the investigation herein presented to the response of multi-storey frame systems. In the following results only the principal components were applied along the X-axis of the buildings ($\theta = 0^o$), thus allowing the actual variation of rotation to be clearly identified without inertial interaction due to secondary excitation (the influence of the orthogonal earthquake component has been considered in further parametric studies not present here).

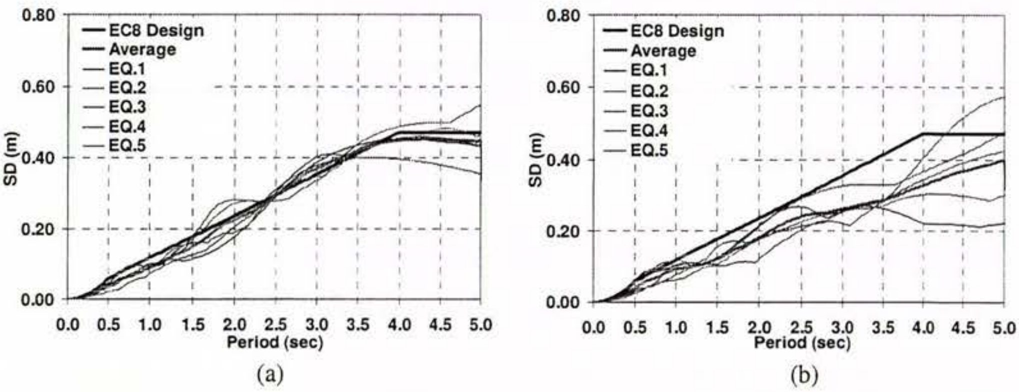


Figure 2. Elastic earthquake displacement spectra ($\xi = 20\%$) (a) principal components (b) secondary components.

4 RESULTS

The inelastic time-history results from the set of frame systems described in the previous section are summarised below. Basic indications of the response in terms of maximum and residual rotations are provided with the intent to identify, at this stage, qualitative trends to be used in subsequent investigations that can define the design issues and more appropriate quantitative values.

4.1 Influence of System Restraint Conditions

In the work presented by Castillo (2004), the influence of the translational elements (quantified by the ratio of radius of gyration of strength to radius of gyration of mass r_{vx}/r_m of the elements parallel to the principal direction of excitation) was investigated (by keeping the strength distribution constant while changing the distribution of mass) and identified as the basic factor in limiting system rotations. The degree of torsional restraint provided by the transverse elements (r_{vz}/r_{vx}) was also considered, however variations in transverse restraint were relatively insignficant.

In this study the X-direction elements (for the unrestrained systems, see Figure 1) and Z-direction elements (for the restrained systems) were shifted such that the radii of gyration of strength in either direction were set at r_{vx} = 6m, 4m, 2m and 0m for the unrestrained systems (a) and r_{vz} = 9m, 6m, 4m and 0m for the restrained systems (b). For each structure the same mass eccentricities were applied for each variation of restraint level. It should be noted that P-Δ effects were not included in these analyses as this would be included as an additional factor in the explicit design approach proposed by Christopoulos et al. (2004a & b).

The dynamic behaviour of these two building typologies provides some interesting comparisons to the traditionally accepted trends of torsional response. Shown in Figure 3 are results in terms of the average maximum, residual and residual/maximum diaphragm rotation for each building type as a function of eccentricity. When considering the maximum rotations, the behaviour is generally as expected with the unrestrained systems exhibiting larger rotations than the restrained counterparts.

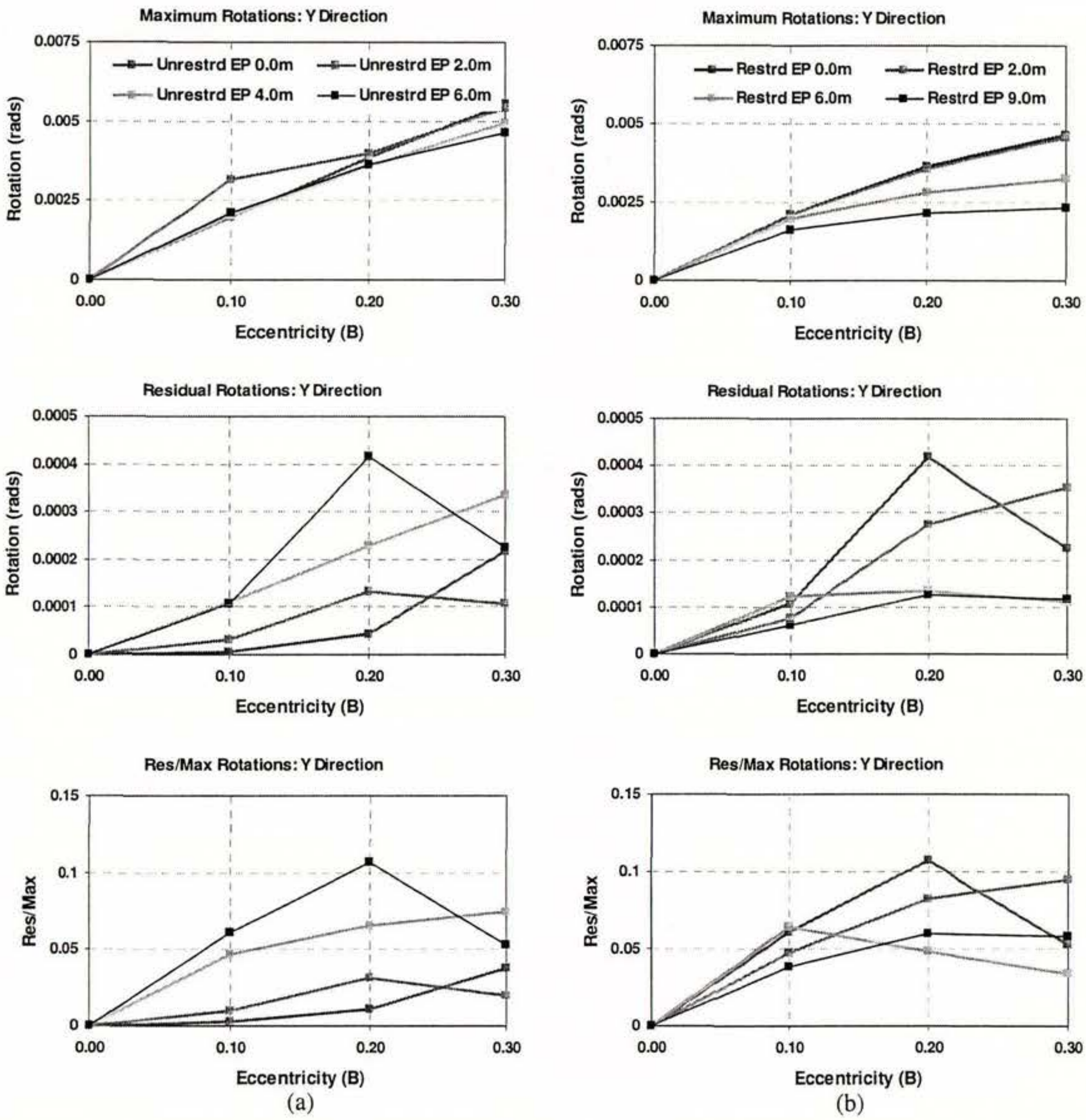


Figure 3. Results indicating the effect of variations in radii of gyration of strength (a) Bilinear Unrestrained frame r_{vx} (b) Bilinear Restrained frame r_{vz} (No P-Δ effects included in analyses)

Attention is now given to the differentiation between restrained and unrestrained buildings (i.e. type (a) or (b)). In the case of the restrained buildings, both column and beam hysteretic action is developed in the global Z-direction frames due to the diaphragm rotations, whereas the unrestrained buildings only exhibit column hysteresis. With the presence of axial loads (both gravity and seismic) in the columns the hysteretic loops will tend to be narrower or smaller. In the case of reinforced concrete inelastic behaviour (represented by a Modified Takeda hysteresis rule) the unloading stiffness will be significantly reduced leading to narrower loops that have an inherent re-centring ability.

Considering columns 1, 4, 6 and 9 (at the ends of the X-axis in the resisting frames) it is evident that the unrestrained systems only develop out-of-plane hysteretic action in the columns, as there are no resisting beams present. These systems can exhibit lower residual deformations or re-centre to a greater extent than the restrained counterparts, even though the unrestrained maximum rotations are clearly larger.

The implication of these observations is that while higher maximum diaphragm rotations are typically expected from torsionally unrestrained and asymmetric buildings, residual deformations might not follow the same amplification trend. From the results presented here it is suggested that for non-perimeter frame (or wall) systems the residual/maximum rotation ratio for unrestrained structures tends to be lower than that for the restrained systems. It is possible that such systems can in fact produce lower residual/maximum plan rotations due to the interaction of low amplitude diaphragm rotation cycles and degrading column stiffness response (in the case of Takeda hysteresis), as well as the lack of beam hysteresis which generally produces greater residual deformations.

Figure 4 shows the average Res/Max results as a function of mass eccentricity and restraint for RC frame behaviour. Clearly the Res/Max ratio results are significantly lower than those seen for EP systems in Figure 3 and for most levels of eccentricity these average values could be neglected, although the scatter as shown in Figure 5a suggests this may be not be satisfactory. It is interesting to note the difference in curve shapes between Figure 4 and Figure 3, with the TK results tending to grow as eccentricity increases, whereas the EP curves exhibit more of a plateau with increasing eccentricity.

A more significant consideration comes from the comparative response between systems with decreasing levels of torsional restraint. As the separation between restraining elements decreases (i.e. lowering the level of torsional restraint or radius of gyration of strength) the displacement demands on the seismic resisting members due to twist are reduced, therefore the resisting elements have lower inelastic demands. Thus, while the extreme corner maximum displacements are amplified by the twist during the excitation, the structural members (now situated closer to the GC of the building) are not subjected to high ductility levels, therefore the final diaphragm rotation is reduced as these control the residual response. Hence even though a system with lower restraint will undergo greater maximum rotational response, the residual response will not necessarily maintain the same proportionality with the increased maximum.

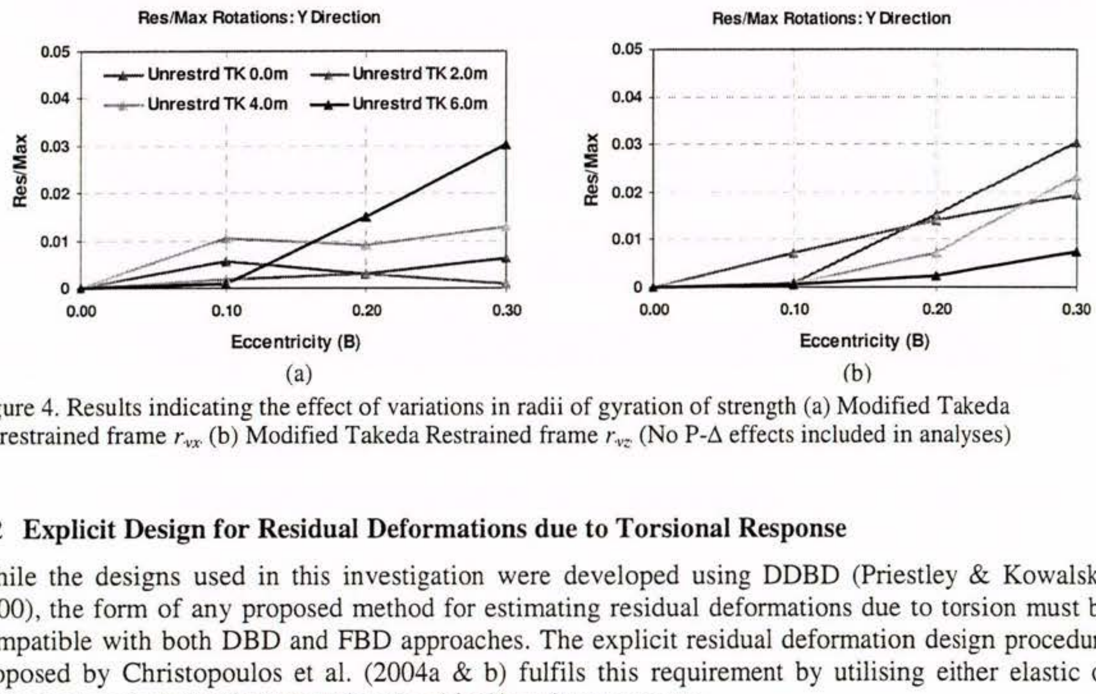


Figure 4. Results indicating the effect of variations in radii of gyration of strength (a) Modified Takeda Unrestrained frame r_{vx} (b) Modified Takeda Restrained frame r_{vz} (No P-Δ effects included in analyses)

4.2 Explicit Design for Residual Deformations due to Torsional Response

While the designs used in this investigation were developed using DDBD (Priestley & Kowalsky 2000), the form of any proposed method for estimating residual deformations due to torsion must be compatible with both DBD and FBD approaches. The explicit residual deformation design procedure proposed by Christopoulos et al. (2004a & b) fulfils this requirement by utilising either elastic or equivalent periods for the translational residual/maximum spectra.

It is tentatively proposed here that the complete equation describing the residual deformation (*RD*) would take the following general form for single-storey structures:

$$RD_{SDOF} \cdot f_{P-\Delta} + \Gamma \cdot RD_{Torsion} \cdot f'_{P-\Delta} = RD_{Total} \tag{1}$$

which can be further defined as:

$$\theta_{SDOF,Max} \cdot \left(\mathrm{Res}/_{Max}\right)_{2D} \cdot f_{P-\Delta} + \Gamma \cdot \varphi_{Tors,Max} \cdot \left(\mathrm{Res}/_{Max}\right)_{3D} \cdot \psi \cdot f'_{P-\Delta} = RD_{Total} \tag{2}$$

where the first component of the design equation represents the residual drift due to translational response as defined in previous studies (Christopoulos et al. 2003, Pampanin et al. 2003, Christopoulos et al. 2004a & b), while the second part defines the additional residual drift due to diaphragm rotations. The reader is referred to the previous studies for further information on the complete design procedure. In this equation $\varphi_{Tors,Max}$ is the expected maximum diaphragm rotation determined by some means (for example Sommer & Bachmann 2005 or Trombetti et al. 2002), which must also indicate whether inelastic torsional response occurs, something which available methods do not currently achieve. ψ is a geometric ratio (in-plan distance from the CM/inter-storey height) that converts the diaphragm rotation to lateral drift at the element in the structure under consideration. To account for the varying level of concurrency between the observed maximum translational and rotational responses (a phenomenon that is highly dependent on the type of system considered), a "Phase Coupling Coefficient" Γ, is introduced. It should be noted that the two P-Δ amplification terms $f_{P-\Delta}$ and $f_{P-\Delta}'$ are defined separately for translation and rotation respectively.

As has been shown above, the value of the Res/Max ratio for diaphragm rotation is effectively dependent on the level of torsional restraint provided. The ratio of pure rotational circular frequency (ω_θ) to pure translational circular frequency (ω_L) provides a simple and efficient way of quantifying the torsional restraint in a dynamic sense. This ratio is defined as γ by Trombetti et al. (2002).

$$\gamma = \frac{\omega_\theta}{\omega_L} \tag{3}$$

By plotting the individual column results, shown as averages in Figure 3 and Figure 4, against the corresponding value of γ for each system (Figure 5) it can be seen that, in torsional systems with $1.0 \leq \gamma \leq 1.25$, the residual/maximum rotation ratios reach a peak, at which point restrained and unrestrained systems have similar results.

If the plots in Figure 5 are termed "Residual/Maximum Rotation Spectra" it is possible to enter the appropriate chart with a calculated value of γ and retrieve the design value of rotational Res/Max, using an appropriately fitted design curve (as suggested by the smoothed mean + 1 standard deviation curve in Figure 5). With this ratio defined, and the maximum rotational response also calculated, the total residual deformation (storey drift) can be found from Equation 2 and compared with the target residual drift (Christopoulos et al. 2004).

In applying Equation 2 consideration should be given to whether it is applicable to augment the residual deformation on one or both sides of the structure (taken to be either side of the CM). This decision will depend on whether the building is expected to develop inelastic action on both sides under coupled response. Force-based design methods would suggest that buildings will generally achieve ductile response in all resisting elements due to the generally high values of design ductility allowed in current design codes. Under displacement-based approaches it has been shown (Pettinga & Priestley 2005) that drift limits will often dictate the maximum allowable ductility developed under a design earthquake. Therefore it is possible that in many cases the design system ductility μ_Δ will be between 2 and 3. In the study by Peruš and Fajfar (2005) it was demonstrated that maximum rotational response will occur when one side of a structure remains within or close to its linear-elastic limit as this induces a significant shift of the centre of stiffness (or rotation) therefore increasing the eccentric distance from the centre of mass.

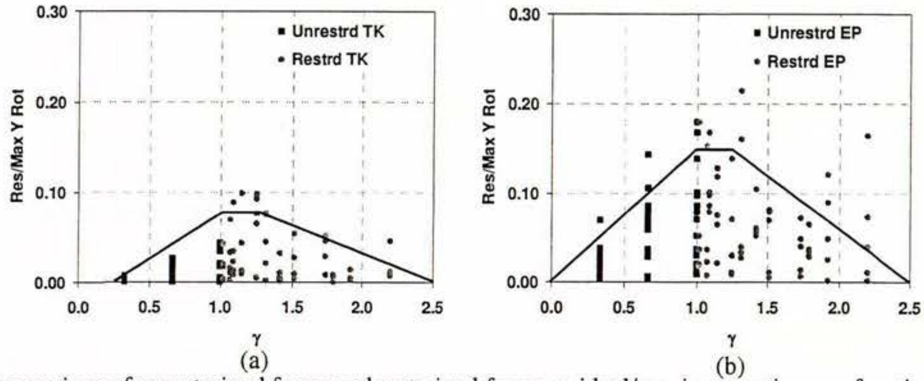


Figure 5. Comparison of unrestrained frame and restrained frame residual/maximum ratios as a function of $\gamma = \omega_\theta/\omega_L$: (a) Takeda hysteresis (b) Bilinear hysteresis.

Thus if the system ductility is limited to low values as suggested above it is expected that elements on one side of the structure may not yield or would not be negatively influenced by the coupled response, while those on the first-yielding side could suffer further inelastic cycles (however not necessarily greater displacements) due the significant rotations induced. It is assumed here that the increased number of inelastic cycles (rather than simply increased maximum ductility achieved) can be more significant in terms of residual drifts.

5 CONCLUSIONS

The preliminary results from an ongoing investigation into the residual deformation response of 3-D structures are presented. The comparative effects of varying torsional restraint in simple prototype mass eccentric frame systems have been highlighted. It has been shown that residual diaphragm rotations can be reduced for systems with a lowered level of torsional restraint due to the reduced demand induced in the resisting elements under coupled response. Further to this the position of potential plastic hinge zones can also play a role in determining the level of residual deformation, with beam plastic hinging tending to exhibit larger hysteretic loops that dominate the system residual deformation behaviour.

Based on these preliminary results and further parametric studies (not included here) a general design approach is suggested that can be included within the explicit design procedure already presented in earlier work. Principally this 3-D component utilises an estimation of the maximum system rotation and a value of residual/maximum rotation determined from a "Residual/Maximum Rotation Spectrum". This value of Res/Max rotation is found as a function of γ, the ratio of elastic fictional pure rotational frequency to pure translational frequency. In a similar manner to the determination of translational residual drift, the additional drift due to P-Δ effects is accounted for by including a scaling factor that amplifies the basic value of residual drift.

Finally an important point is highlighted regarding the development of inelastic action in parallel resisting elements in an asymmetric structure. It is noted that for buildings in which maximum allowable drift demand controls the ductility to be developed, certain systems may not exhibit inelastic action in all elements either side of the centre-of-mass, giving potential for a significant shift of the centre-of-stiffness that can lead to increases in the system eccentricity, and thus diaphragm rotations.

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A PROBABILISTIC FRAMEWORK FOR PERFORMANCE-BASED SEISMIC ASSESSMENT OF STRUCTURES CONSIDERING RESIDUAL DEFORMATIONS

Uma S R[1], Stefano PAMPANIN[2] and Constantin CHRISTOPOULOS[3]

SUMMARY

Recent advances in performance based design and assessment procedures have highlighted the importance of considering residual deformations in addition to maximum deformations as a complementary damage indicator. A combined 3-dimensional performance matrix, where maximum and residual deformations are combined to identify performance levels coupled with various seismic intensity levels is presented within a probabilistic formulation of a performance based assessment procedures. Combined fragility curves providing the probability of exceedence of performance levels defined by pairs of maximum-residual deformations are derived using bivariate probability distributions, due to the statistical dependence of the two parameters.
Numerical examples on equivalent SDOF systems with extensive non-linear time history analyses under a properly selected suite of earthquakes are performed to derive the fragility curves for various performance levels. The effects of hysteretic systems and strength ratios on fragility curves are examined. The significance of accounting for residual deformations in addition to maximum deformation indices when evaluating the actual performance level is confirmed by using joined fragility curves. In conclusion, for a given strength ratio and performance level, joined fragility spectra are generated for a range of effective secant periods of SDOF systems providing a measure of confidence in achieving the targeted performance.

1. INTRODUCTION

Performance Based Earthquake Engineering (PBEE) approaches typically assesses the performance of a structure using one or multiple structural response indices, usually based on maximum responses. Recent developments in performance-based design and assessment concepts [Pampanin et al., 2002 Christopoulos and Pampanin, 2004], have highlighted the limitations and inconsistencies related to these traditional approaches. Reports from past earthquake reconnaissance observations, from shake table tests, as well as from analytical studies, indicate that most structures designed according to current codes will sustain residual deformations in the event of a design level earthquake, even if they perform exactly as expected.

Assessing the residual deformations in the structure in the event of a major earthquake is very important with regard to the difficulty and cost associated with the straightening of structures [Priestley, 1993]. A number of researchers [MacRae and Kawashima, 1997; Borzi et al., 2001; Christopoulos et al., 2003] have identified the post yielding stiffness as the main parameter influencing the residual deformations of non-linear Single Degree of Freedom (SDOF) oscillators. A first attempt to introduce limits on residual deformation/drift in design guidelines or code provisions is found in the 1996 Japanese seismic design specifications for highway bridges, which, as reported by Kawashima [1997], imposes an additional design check for important bridges in terms of residual displacements which are required to be smaller than 1% of the bridge height. In recent draft guidelines for performance evaluation of earthquake resistant reinforced concrete buildings under preparation by the

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Architectural Institute of Japan (AIJ), limits on residual crack widths are tentatively indicated and associated to ranges of maximum drift/ductility and damage level.

A residual deformation damage index (RDDI), which measures the degree of permanent deformations and drifts of SDOF or MODF structures, has been proposed in Pampanin et al., [2002, 2003] and Christopoulos et al. [2003] as an additional damage indicator to fully quantify the performance level of buildings under seismic loading. In these studies, as part of a more refined framework for performance-based design and assessment procedures, the concept of joined performance levels, combining maximum and residual deformation, coupled with seismic intensity in the form of 3-dimensional performance matrix has been suggested.

More recently, a direct displacement-based design approach which includes an explicit consideration of the expected residual deformations has also been proposed by Christopoulos and Pampanin [2004]. Building on the aforementioned framework, extensive numerical analyses have been carried out by Garcia and Miranda [2006] to propose an empirical relationship to evaluate the ratios of residual displacement demand to the peak elastic displacement demand for SDOF systems with known strength ratios. It has been observed that residual displacement ratios exhibit larger levels of record-to-record variability when compared to peak inelastic displacements. With the recent developments of probabilistic approaches for performance based earthquake engineering, considering the uncertainties on the seismic hazard and on the structural capacities, preliminary suggestions to describe the design objectives in the form of fragility curves representing the probabilities of exceedence of different damage states for various seismic intensity levels have been given by the authors [Uma et al, 2006].

In this contribution, as part of the on-going development for a refined framework for performance based seismic design and assessment procedures, a conceptual description of probabilistic formulation of the performance based matrix is briefly given. Non-linear dynamic analyses are carried out on a SDOF system under a properly chosen suite of earthquake records to derive fragility curves where combined maximum-residual performance objectives associated with a targeted probability of exceedence are established. Also, the effects of hysteretic systems and strength ratios on fragility curves are studied and suggestions for design/assessment are proposed. Joined fragility spectra are also derived and suggested to assess the confidence in achieving the desired performance level.

2. PERFORMANCE DESIGN OBJECTIVE MATRIX BASED ON MAXIMUM AND RESIDUAL DEFORMATIONS

The objective of Performance Based Seismic Engineering (PBSE) is to design, construct and maintain facilities with better damage control. A comprehensive document has been prepared by the SEAOC Vision 2000 Committee [1995] that includes interim recommendations. The performance design objectives couple expected or desired performance levels with levels of seismic hazard as illustrated by the Performance Design Objective Matrix shown in Fig.1.

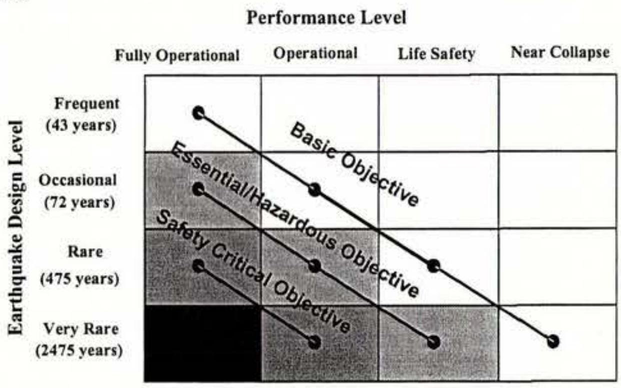
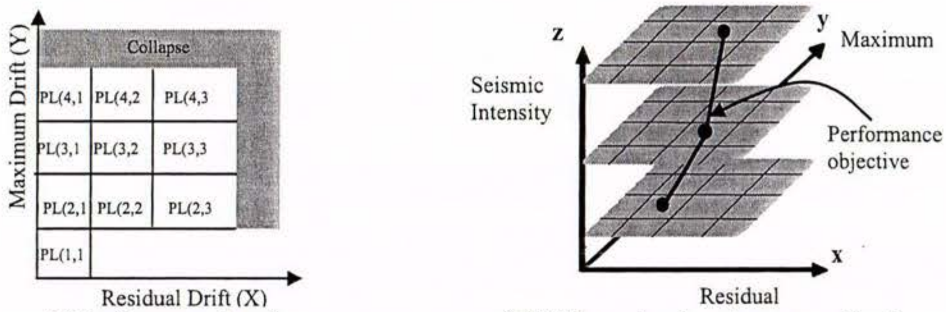


Figure 1 Seismic Performance Design Objective Matrix [SEAOC Vision 2000, 1995]

Recognising the importance of accounting for residual deformations in assessment of the actual performance of structures, for a given seismic intensity, the aforementioned joined performance levels [Pampanin et al., 2002] can capture the joined occurrence of maximum and residual responses. As a result, 2-D performance domain (Fig. 2a, X-Y plane) can be used consisting of Performance Levels, PL(i,j), defined by the combination of Maximum Deformation or Drift, MD, (index i) and Residual Deformation or Drift, RD, (index j). By accounting for the effect of seismic intensity, a 3-dimensional performance matrix (Fig. 2b) can be visualised as a set of pre-defined joined performance domains ("masks") for different seismic intensity level, IM (Z-axis). It should be noted that for a given value of maximum response parameter the performance levels would thus be poorer when combined with higher level of residual responses leading to increased damage and repair costs.

Analogous to the Performance Design Objective Matrix developed by the SEAOC Vision 2000 document, alternative Performance Objectives associated with different structural systems can be defined within the 3-D performance matrix by connecting a set of performance levels/domains PL(i,j) belonging to different intensity levels (Fig. 2b).



(a) Performance domain (b) 3-Dimensional performance objective matrix

Figure 2 Framework for Performance Based Design and Assessment Approach [Pampanin et al., 2002]

3. PROBABILISTIC FORMULATION OF PERFORMANCE BASED MATRIX CONCEPT

In principle, either a deterministic or probabilistic approach could be used within a performance based design or assessment procedure, with preference to the latter approach when a particular level of confidence of achieving performance objective is of interest. More recently, a probabilistic framework for performance-based design and assessment evaluation has been proposed by the Pacific Earthquake Engineering Research Center (PEER) [Cornell, 2000]. The PEER performance-based design framework utilizes the total probability theory to de-aggregate the problem into several interim probabilistic models (namely seismic hazard, demand, capacity and loss models), to account for the randomness and uncertainty in a more rigorous way. The basic and necessary assumption is that all interim models are statistically independent. The mean annual frequency of a decision variable (DV) can be expressed within the frame work of performance based design as

$$\nu(DV)= \iiint G\langle DV|DM\rangle \; |dG\langle DM|EDP\rangle \; |dG\langle EDP|IM\rangle \; |d\lambda(IM) \tag{1}$$

It should be noted that all the interim models are handling only one parameter conditioned on one other parameter. In this study, the development of demand and corresponding capacity models with reference to the performance based matrix concept is discussed. The demand models reported in literature have typically consisted of prediction of the probability of exceeding a given value of one engineering demand parameter (EDP) for a given level of Intensity Measure (IM). When implementing the concept of a joined performance-based matrix, the performance levels are defined using a pair of EDPs, i.e. residual and maximum deformations. Hence, a new Probabilistic Seismic Demand Model (PSDM) relating the effects of the selected IM to two EDPs has to be developed.

The probabilistic assessment of seismic structural performance of a given structure for a given seismic environment is performed using suitable *probabilistic seismic demand model* (PSDM)s which represent the relationship between EDPs and ground motion IMs [Jankovic and Stojadinovic, 2004]. Considering the parameters of the PSDM as continuous Random Variables (RV), the uncertainty involved in the prediction of the values of EDPs can be accounted for by associating suitable probability distributions to the RVs. Let us consider three RVs, namely X, Y and Z corresponding to residual drift (RD), maximum drift (MD) and seismic intensity measure (IM), respectively. Let their individual (marginal) Probability Density Function (PDF) be $f_X(x)$,

$f_Y(y)$ and $f_Z(z)$. The function $f_{X,Y}(x,y)$ is geometrically represented in 3 dimensions by a surface above the (x-y) plane with RD and MD along the x and y axes and whose range is the set of probability values corresponding to the ordered pairs of *(x,y)* in its domain as shown in Fig 3. At any seismic intensity level, the probability of joint occurrence of the two RVs (X,Y) with values corresponding to a performance level, say PL(2,2) could be obtained from the joint PDF and is represented by volume *beneath* this surface as illustrated in Fig.3.

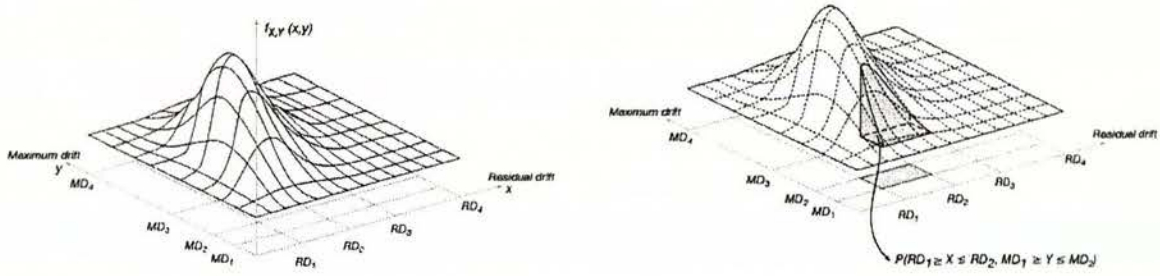


Figure 3 Joint Probability Density Function over a Performance Domain

As reported in previous studies in literature [Pampanin et al, 2002], RD and MD have shown a different degree of correlation at various intensity levels, thus impairing the hypothesis of statistically independent variables. Single bivariate lognormal distribution has been used as joint PDF to describe the joint occurrence of a pair of RD and MD over a performance domain for a given intensity level. It is based on the observed trends of EDPs, typically used in PSDMs that usually follow a lognormal distribution. A bivariate log-normal distribution for the joint distribution of residual drift (X) and maximum drift (Y) with the joint PDF may be written as

$$f_{X,Y}(x,y) = \frac{0.5}{xy\pi\varsigma_X\varsigma_Y\sqrt{1-\rho^2}} *$$
$$\exp\left\{-\frac{0.5}{1-\rho^2}\left[\frac{(\log x-\lambda_X)^2}{\varsigma_X{}^2} - \frac{2\rho(\log x-\lambda_X)(\log y-\lambda_Y)}{\varsigma_X\varsigma_Y} + \frac{(\log y-\lambda_Y)^2}{\varsigma_Y{}^2}\right]\right\} \tag{2}$$

Where $\lambda_X, \lambda_Y, \varsigma_X$ and ς_Y are the location and scale parameters of the marginal PDF of X (residual drift) and Y (maximum drift), respectively. The parameter ρ forms a linear correlation coefficient between the two variables.

4. DEFINITION OF PERFORMANCE OBJECTIVES ACCORDING TO THE 3-D PERFORMANCE MATRIX CONCEPT: A PROBABILISTIC APPROACH

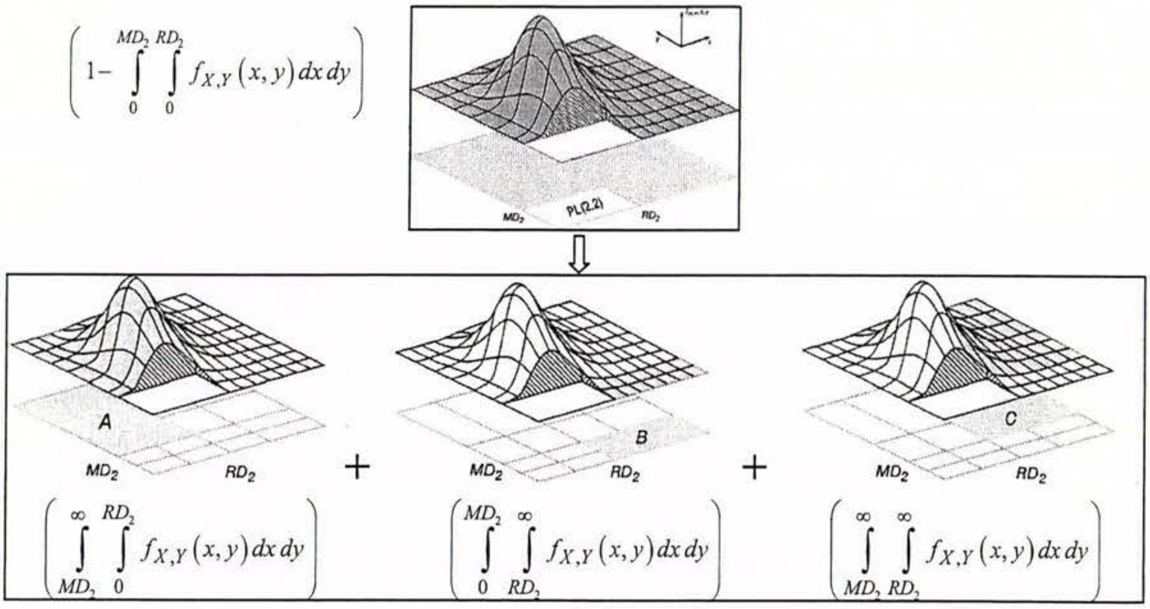
4.1 Probabilistic procedure adopted on 3-dimensional performance-based matrix

A PSDM is appropriately selected and seismic response analyses are carried out for the chosen structural system for a suite of earthquake records varying the levels of IM. The EDPs the residual and maximum drifts at every level of IM are analysed for their statistical parameters to describe the joint PDF. These data pairs correspond to a single 2-D performance domain.

The probability of achieving a PL(i,j) specific to certain domain of RD and MD is obtained by performing double integration over the joint PDF as in Eq. 3, with respective values of the variables as upper and lower limits of integration. For example, the probability associated with PL (2,2) is given by

$$\int_{MD_1}^{MD_2}\int_{RD_1}^{RD_2} f_{X,Y}(x,y)\, dxdy \tag{3}$$

The probability of exceeding a generic PL(i,j), for example PL(2,2), is given by the volume under shaded portion of the surface area as shown in Figure 4 ,which may also be expressed as 1 minus the probability of reaching or being within PL(2,2). At this stage, it may be of interest to know the contribution to the probability distribution by alternative pairs of RD and MD. As shown in Fig. 4, the zone A may be interpreted as contribution mostly governed by MD, zone B as that mostly governed by RD and zone C as that governed by both parameters.



$$\left(1-\int_{0}^{MD_2}\int_{0}^{RD_2} f_{X,Y}(x,y)\,dx\,dy\right)$$

$$\left(\int_{MD_2}^{\infty}\int_{0}^{RD_2} f_{X,Y}(x,y)\,dx\,dy\right) + \left(\int_{0}^{MD_2}\int_{RD_2}^{\infty} f_{X,Y}(x,y)\,dx\,dy\right) + \left(\int_{MD_2}^{\infty}\int_{RD_2}^{\infty} f_{X,Y}(x,y)\,dx\,dy\right)$$

Figure 4 Probability of exceedence of PL(2,2) and the contributions from the response parameters

Joint PDF enables the computation of total probability of exceedence for PL(i,j) on a performance domain associated with a given level of IM. A cumulative distribution of probability of exceedence at increasing intensity levels gives fragility curves. Performance objectives connecting various performance levels with increasing seismic hazard levels can be established using these fragility curves.

5. NUMERICAL EXAMPLES ON SDOF SYSTEMS

The performance evaluation procedure according to a probabilistic approach is herein illustrated with numerical examples on equivalent Single Degree of Freedom (SDOF) systems. The exercise essentially includes selecting ground motion records, performing seismic response analyses and developing fragility curves for performance levels representing damage limit states. A spectrum of fragility curves representing a particular performance level can be established by choosing SDOF oscillators with a range of periods. A parametric study has been conducted with respect to hysteretic models and strength ratios defined as the ratio of the base shear at yielding to the building weight.

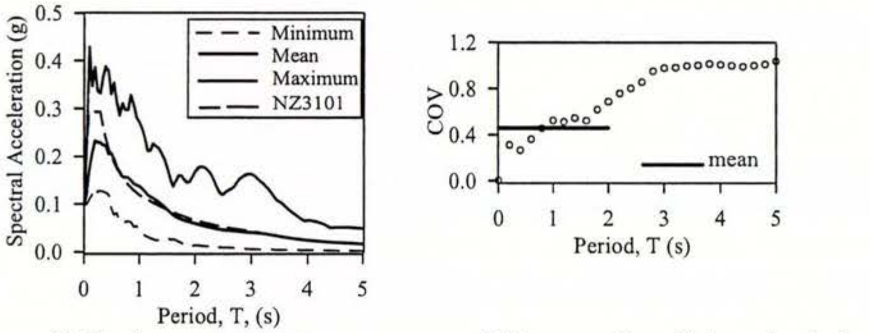
5.1 Properties of Single Degree of Freedom Systems

An equivalent SDOF system representing a 4 storey reinforced concrete building designed according to a displacement based design approach with a target maximum inter-storey drift of 2.5% and a peak spectrum acceleration of 0.5g [Priestely, 1998] has been considered. The dynamic properties of the equivalent SDOF systems are: (i) initial elastic period, T_1 = 1.11s; (ii) Strength ratio S_r between the base shear at yielding, F_y = 1040 kN and the building weight, W of 4000 kN is 0.26; (iii) Effective heights, H_{eff} = 9 m; (iv) Effective weights (first mode), W_{eff} is 3333 KN [Pampanin et al., 2002]. The SDOF systems are modelled in Ruaumoko [Carr, A.J., 2005] based on a lumped plasticity approach.

5.2 Selection of Ground Motions and Representative Intensity Measure

A total of thirty earthquake ground motions were utilised in this study. They were extracted from two sources: the database used by Pampanin et al. [2002] and the Pacific Earthquake Research database [PEER, 2000]. The records represent magnitude ranging from 6.5 to 7.2, closest distance to fault rupture varying from 15 km to 30 km and soil category C and D (according to NEHRP provisions [1997]). The response spectra with 5% damping for each 30 earthquake records scaled to 0.1g is shown in Figure 5a. A significant degree of record to record variation can be observed with respect to the mean spectral curve. The degree of variation is plotted as lognormal coefficient of variation as shown in Figure 5b. The average coefficient of variation is 0.46 for periods shorter than 2 s. It can be noted that the mean spectrum is in good agreement with the NZS 1170.5 (2004) code design spectrum for PGA of 0.1g with soil category C except for very short periods, less than 0.5 s.

In this numerical study, the spectral acceleration (S_a) corresponding to the initial period of the building (T_1) is chosen as the intensity measure (IM) to satisfy the statistical independence of the hazard model with respect to the magnitude (M) and the distance (R) in predicting the engineering demand parameters. This has been verified by conducting a multivariate linear regression analysis [Mackie and Stojadonivic, 2003]. It was observed that the regression coefficients corresponding to the magnitude and distance variables were not as significant as the one corresponding to the IM, thus ensuring the sufficiency of the model to relate independently $S_a(T_1)$ with EDPs. Similar studies on Equivalent SDOF system representing an eight story building with spectral velocity S_v has been reported by the authors elsewhere [Uma et al., 2006].



(a) Elastic response spectra (b) Lognormal coefficient of variation

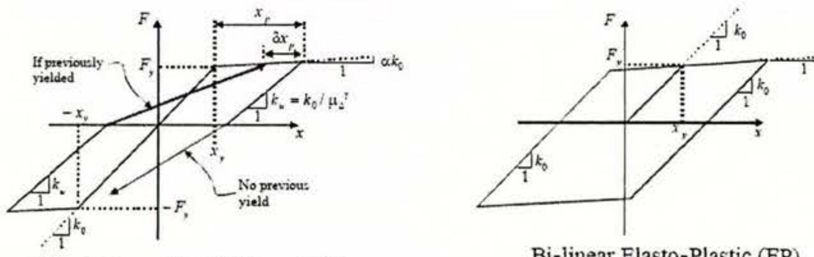
Figure 5 Response spectra with 5% damping for the earthquake records normalised to 0.1g

5.3 Definition of Limit States for Engineering Demand Parameters

As mentioned, structural and non-structural damage limit states or performance levels have been typically related to maximum transient responses. However, recent publications have emphasized the need to check the permanent (residual) deformations in structures and have suggested tentative residual drift limits based on percentage of the maximum expected drift of the structure [NEHRP, 1997, Kawashima, 1997, FEMA 356, 2004]. In this study, referring to the previous research work [Pampanin et al., 2002] and the draft guidelines of AIJ [2004], tentative values for the limit states based on residual drift, RD, are taken as 0.2%, 0.4% and 0.6% and 1.0% while, more traditional values for the limit states based on maximum drift, MD, are considered as 0.5%, 1.0%, 2.0% and 4.0%. The limit states can be typically referred to as "Serviceability", "Repairable Damage", "Irreparable Damage" and "Collapse prevention".

5.4 Seismic Response Analysis

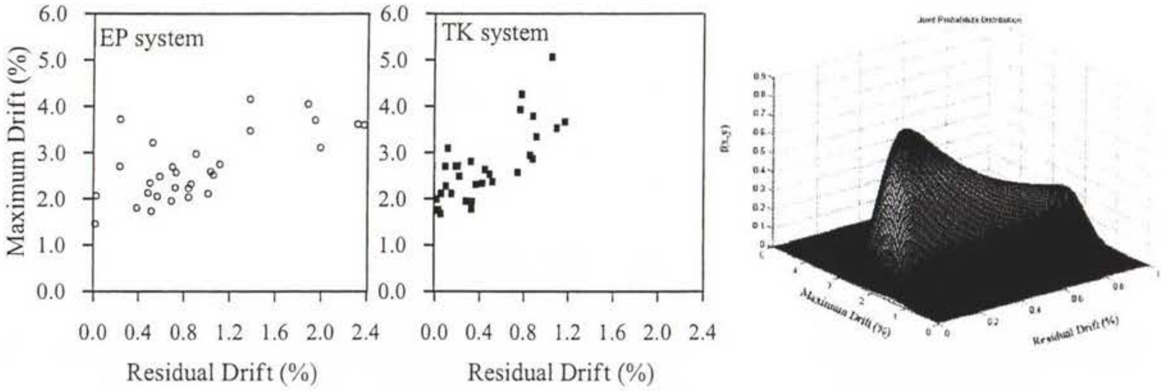
A series of inelastic time history analyses using the selected suite of earthquake records was performed on SDOF systems based on a lumped plasticity approach for two hysteretic models namely elasto-plastic (EP) and Takeda (TK) as shown in Fig. 6. The performance of the building in terms of maximum and residual drift ratio (with respect to the effective height) is studied at various seismic intensity levels by scaling up the IM ($S_a(T_1)$)from 0.2g to 2g.



Takeda Degrading Stiffness (TK) Bi-linear Elasto-Plastic (EP)

Figure 6 Hysteretic Models used in the Analyses

The analyses are performed for 30 records for a chosen level of intensity measure and repeated for 10 levels of intensity measure. The distribution of RD and MD at spectral acceleration of 0.8g for EP and TK systems are shown in Fig 7a,b and the joint PDF for EP system is shown in Fig 7c. The lower values of residual with less scatter is shown by TK systems compared to EP systems.



(a) Scatter plot (EP); (b) Scatter plot (TK) (c) Joint probability distribution (EP)

Figure 7 Distribution of residual and maximum drift at $S_a\,(T_1)$= 0.8g

5.5 Development of fragility curves

The residual and maximum drift ratios of SDOF systems were evaluated for all 30 records at each level of intensity measure, IM, and the corresponding statistical parameters were derived The distribution of RD and MD at each intensity level is described as a bivariate lognormal joint PDF using their respective lognormal mean, lognormal standard deviation and their correlation coefficient. The total probability of reaching or exceeding a desired performance level for a given intensity can be computed using the corresponding damage limit states as integration limits, as described in section 5.1. A smooth fragility can be fitted to the computed probability of exceedence values assuming a lognormal distribution.

5.5.1 Significance of including residual deformations as a complementary damage indicator

As mentioned, residual deformation has been proposed by Pampanin et al., [2002] as a complementary damage indicator parameter in addition to maximum deformation indices for assessing the actual performance level of a structure. In other words, the PL defined by a given maximum drift ratio limit with lower residuals might represent a lower damage state when compared to the PLs corresponding to same maximum drift limit but larger residual drift limits. The significance has been illustrated in Figure 8, where fragility curves obtained for PLs corresponding to maximum drift limits (for i= 3 and 4) combined with residual drift limits (for j=1,2,3) are presented. It can be seen that for a chosen intensity level, i.e. Sa = 0.6g, the probability of exceedence '(d)' of PL(4,1) is higher than that corresponding to '(b)' of PL(3,2) and '(c)' of PL(3,3). It should in fact be noted that for a given intensity level, the fragility curve with higher probability of exceedence indicates a lower level of damage whereas the curve with lower probability of exceedence represents a higher level of damage. Thus, it is evident that, although being subjected to similar maximum drift demands, the systems should be assigned substantially different levels of performance, depending on the value of the residual response indices.

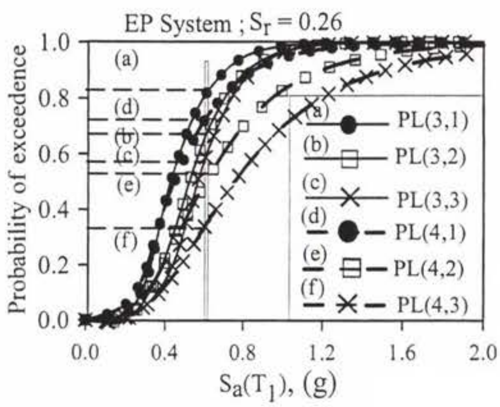


Figure 8 Influence of residual drift on fragility curves

5.5.2. *Effect of hysteretic systems on joined fragility curve*

The nonlinear systems were assumed to exhibit two hysteretic behaviours: the EP and TK models. Both these systems were assigned a zero post yield stiffness ratio. The hysteresis coefficients describing the unloading and reloading stiffness behaviour of a TK model (typically known as α and β) were taken as 0.3 and 0.2 respectively. Figure 9 shows the probability of exceedence of PL(4,2) for the two systems. It can be noted that EP systems show higher probability of exceedence than TK systems regardless of the intensity level. Fig 9 also illustrates the contributions to the total probability of exceedence, referred to A, B and C as per Fig. 4, showing some difference in the behaviour of the two systems. The contributions of residual (B zone) and combined max. and res. (C zone) are in fact predominant in both EP and TK system, whereas some contribution from the maximum drift (A zone) can be noted in the TK system for higher level of IM. From this, the influence of the response parameters to the probability of exceedence of a PL can be recognised.

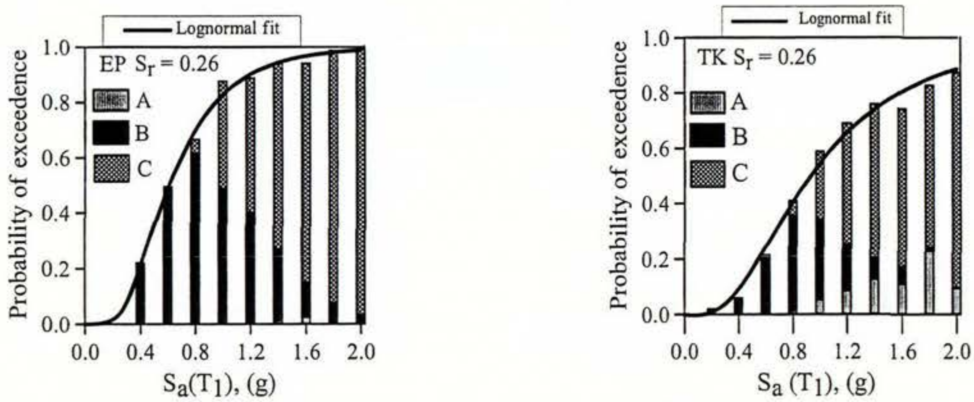
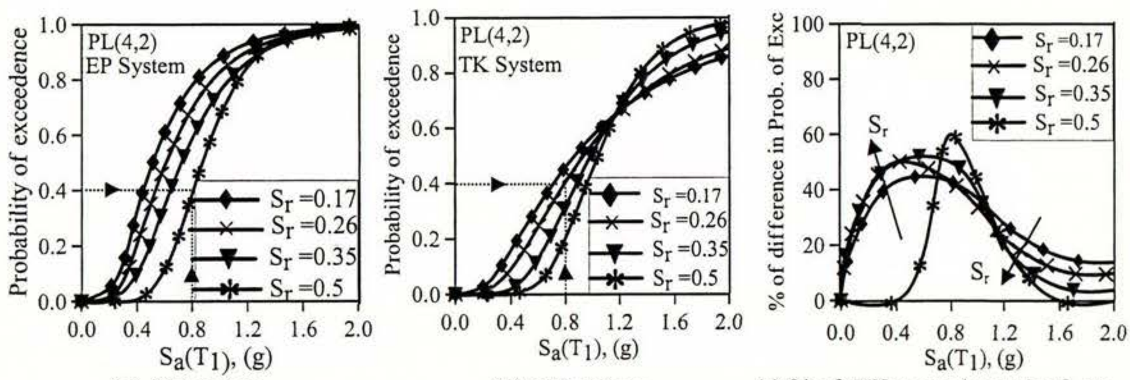


Figure 9 Contribution "zones" to the total probability of exceedence of PL(4,2) for EP and TK systems

5.5.3. *Effect of strength ratios on fragility curve*

The effects of a variation of the system strength ratio, defined as yield strength normalised by the effective weight of the SDOF system, on the total probability of exceedence of performance levels has been investigated. Four strength ratios were considered for the SDOF systems keeping the dynamic and structural properties (i.e mass, damping ratio and fundamental period) the same. Figure 10 (a and b) show the fragility curves for PL (4,2) for the two hysteretic systems considered. In general, higher probabilities of exceedence are observed for lower strength ratios up to a critical intensity level, after which, the probability of exceedence increases as the strength ratio increases. This critical point is observed at a much lower intensity level in the case of TK systems than in the case of EP systems.



(a) EP system **(b) TK system** **(c) % of difference in prob.of .exc**

Figure 10 Effect of strength ratio on fragility for PL (4,2)

A meaningful interpretation can be made from these fragility curves with regard to the required strength ratios for the two hysteretic systems to achieve the same targeted probability of exceedence at a given design intensity level. For example, at a chosen level of intensity, say 0.8g, and a targeted probability of exceedence of 40% for the PL(4,2), the EP system should be designed for a strength ratio of 0.5 whereas the TK system requires only a strength ratio of 0.26. Figure 10 c shows the percentage of variation of the difference in probability of exceedence of two systems with respect to EP system for varying intensity levels. This quantity decreases as the intensity level increases for a certain strength ratio. It can be observed that for all the strength ratios considered, at intensity levels, e.g. 0.4g-0.8g, the difference in probability of exceedence between EP and TK systems is approximately 50%. At an intensity level of 1.2g, it is approximately 25%. At higher intensity levels, the difference is less than 15%. Hence, if PL(4,2) is the targeted PL, the advantage of designing a TK system with a lower strength ratio than an equivalent EP system is more significant at lower intensity levels.

5.5.4. *Fragility spectrum for performance levels based on combination of maximum and residual*

Christopoulos et al., [2004] suggested a modified direct displacement based design method which includes an explicit consideration of residual deformations in the early stages of the design procedure. Inelastic (ductility constant) design spectra based on residual/maximum drift ratios as a function of effective secant period were also derived. In a probabilistic formulation, a joined *'fragility spectrum'* generated for a range of effective periods for different performance levels, defined based on a combination of maximum and residual, would enable the designer to target a performance with certain confidence corresponding to the effective period of the system. Figure 11 shows the fragility spectrum for PL(3,3) for a constant ductility level of 4.

5.5.5 *Definition of performance objectives within probabilistic approach*

In a design or assessment phase, the performance objectives can be obtained by connecting the performance levels with targeted probabilities of achieving them. Fig. 11 shows the visualization of typical performance objectives when adopting fragility curves. A major increase in the confidence of the design could be for example achieved by targeting a defined level of probability of exceedence of different performance levels for increase levels of intensity. Within a complete probabilistic formulation (including the seismic hazard model), a "uniform risk" design approach could be suggested to be followed, consisting of targeting the same probability of exceeding different PLs belonging to a predefined performance objective, as shown in a solid line, UR, in Fig. 12. Alternatively, a variable level of acceptable probability of exceedence (i.e. possibly referred to as "multiple risk" approach in a general formulation) associated to different intensity and performance levels can be adopted in the design phase (as indicated with a dashed line, MR). For example, the MR curve connects PL (2,2), PL(3,3) and PL(4,4) with respective probabilities of achieving them of 40%, 30% and 30%.
It would be thus possible to define and target performance objectives with the associated probability of occurrence by connecting various performance levels with increasing seismic intensity within the 3-D performance matrix framework.

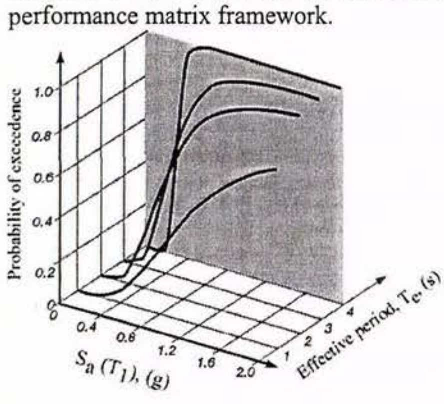


Figure 11 Joined MD-RD fragility spectrum (PL(4,2))

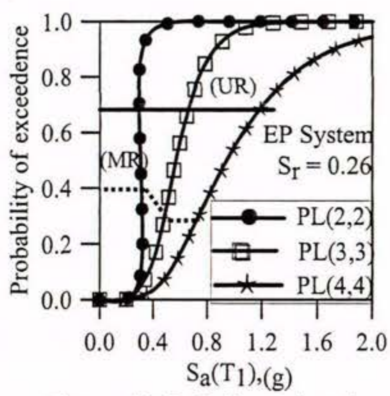


Figure 12 Definition of performance objectives using fragility curves

6. CONCLUSIONS

A probabilistic formulation of a performance matrix combining maximum and residual deformation to define performance levels and performance objectives at increasing level of seismic intensity has been presented. The joint occurrence of residual and maximum deformations within a chosen performance domain is described by a bivariate lognormal probability density functions. Fragility curves representing the probabilities of achieving or exceeding different maximum-residual performance levels are derived. Numerical examples on SDOF systems confirmed that the contributions of the maximum or residual response parameters to the total probability of exceedence of a performance level, PL(i,j) are largely influenced by the hysteresis models. Given the intensity level, the EP systems display a higher probability of exceedence of a PL governed by RD whereas the TK systems show lower probability of exceedence governed by MD. The amount contributed by each response parameters would be an important factor to suggest a mitigation strategy in a design phase as well as a suitable retrofitting intervention to achieve higher performance. The effect of strength ratios on the two hysteretic models with different targeted performance levels is currently being further examined to fully assess the advantages of designing a TK system with a lower strength ratio than the corresponding EP systems at different intensity levels.

In conclusion, preliminary suggestions for a "uniform risk design" (or controlled "multiple risk") approach, whereby targeted probability of exceeding predefined performance levels, at increasing intensity levels, are considered in the design phase, have been given. The concept of a fragility spectrum for various performance levels is tentatively introduced as useful tool within a probabilistic approach of performance-based seismic design/assessment of structures considering residual deformations.

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MITIGATION STRATEGIES AND SOLUTIONS

TASK 2 - Alternative mitigation strategies (not relying on post-tensioning)

Pettinga, J.D., Christopoulos, C., Pampanin, S., Priestley, M.J.N.

"Effectiveness of Simple Approaches in Mitigating Residual Deformations in Buildings", Earthquake Engineering and Structural Dynamics, submitted for publication, under revision

Pettinga, D., Pampanin, S., Christopoulos, C., Priestley, M.J.N

Accounting for Residual Deformation and Simple Approaches to Their Mitigations. 1st ECEES, Geneva, Switzerland, Sept, 2006, paper n. 343

First European Conference on Earthquake Engineering and Seismology
(a joint event of the 13th ECEE & 30th General Assembly of the ESC)
Geneva, Switzerland, 3-8 September 2006
Paper Number:

ACCOUNTING FOR RESIDUAL DEFORMATIONS AND SIMPLE APPROACHES TO THEIR MITIGATION

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SUMMARY

Recent developments in performance-based seismic design and assessment approaches have emphasised the importance of properly assessing and limiting the residual (permanent) deformations, typically sustained by a structure after a seismic event, even when designed according to current code provisions. Recent investigations have led to a proposed Direct Displacement-Based Design (DDBD) approach which includes an explicit consideration of the expected residual deformations accounting for 2-dimensional and MDOF effects. Having estimated the possible residual deformations in a structure, it remains to implement specific design features to reduce them to an acceptable level. Previous studies have identified post-yield stiffness as being critical to residual deformation behaviour, therefore a series of simple approaches are proposed to increase this element and system parameter. These methods do not utilise re-centring post-tensioned technology. First, the effects of changes in material stress-strain behaviour and section design in the primary seismic-resisting system are considered, and then the design and introduction of a secondary elastic frame to act in parallel with the primary system is demonstrated. Using moment-curvature and non-linear time-history analyses, the proposed approaches are shown to be effective at achieving their intended goal of residual deformation reduction.

1. INTRODUCTION

As part of developing performance-based design and assessment concepts, residual deformations are accepted as being important in the overall definition of adequate structural response to earthquake demands. It is evident from the growing number of researchers contributing to this field of study that the assessment and mitigation of residual deformations remains one of the principal topics which needs to be addressed if performance-based design is to be fully defined and applied in practice.

Recent investigations [Christopoulos et al., 2003; Christopoulos and Pampanin, 2004; Ruiz-Garcia and Miranda, 2006; Pettinga et al., 2006a] have advanced the understanding of residual displacement behaviour and have led to proposals for design methods that estimate and explicitly account for permanent deformations.

With the possibility of quantifying the level of residual deformation in a structure, it remains for the designer to reduce (if necessary) these final displacements such that the building meets the appropriate performance targets. In this contribution, a series of approaches to achieving such reductions are considered, with the aim in each case

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being to increase the post-yield stiffness ratio of individual elements and/or the seismic resisting system as a whole. None of the suggested mitigation measures involve the application of post-tensioned elements or external re-centring devices which have been shown to be very effective at reducing or completely removing residual displacements. The approaches presented here represent examples of simple changes that use common construction techniques and design options. They involve changes in (a) material stress-strain behaviour, (b) section design, or (c) design and implementation of secondary seismic systems intended to remain elastic under maximum response. The results presented here will focus on the comparative response to one earthquake ground motion, which is representative of general trends found for a suite of records presented elsewhere [Pettinga et al., 2006b].

2. ANALYTICAL APPROACH

To demonstrate the effect on maximum and residual displacements both moment-curvature and inelastic static and dynamic analyses were carried out. The use of moment-curvature analyses allowed the changes in section-level behaviour to be investigated and where necessary produced the data required for modelling member hysteretic action in the inelastic dynamic models. The program Response-2000 [Bentz and Collins, 2000] was used for the section modelling while a lumped plasticity approach [Carr, 2005] was generally used for the dynamic analyses. In some cases finite-element models using fibre-elements [Antoniou and Pinho, 2005] were implemented in order to accurately observe the dynamic response to specific reinforcing changes within a section.

A vertically and horizontally regular four-storey building (Figure 1) representative of a low-rise commercial property was designed as a reinforced concrete (RC) or steel moment-resisting frame (MRF), or buckling-restrained braced frame (BRBF) in accordance with the New Zealand seismic loading provisions [NZS1170.5:2004] for a Zone factor of 0.4 and deep or soft soil (type D) conditions. To satisfy a maximum drift limit of 2.5% a consistent design ductility (μ_Δ) of 3.5 was used for the reinforced concrete and steel moment-resisting frame buildings (with this ductility value the steel building drifts met or slightly exceeded 2.5%, and the reinforced concrete frame was within 2.5% drift), while the buckling-restrained braced frame was designed to a μ_Δ of 6 as drift limits were not critical. No allowance was made for near-fault effects, however the set of verification records suggested for use with the New Zealand code were representative of near-field events as these are recognised as producing greater residual deformations [Kiggins and Uang, 2006]. The results shown here will focus on the comparative response of the analytical models to the Caleta de Campos (N) record from the 1985 Michoacan earthquake in Mexico.

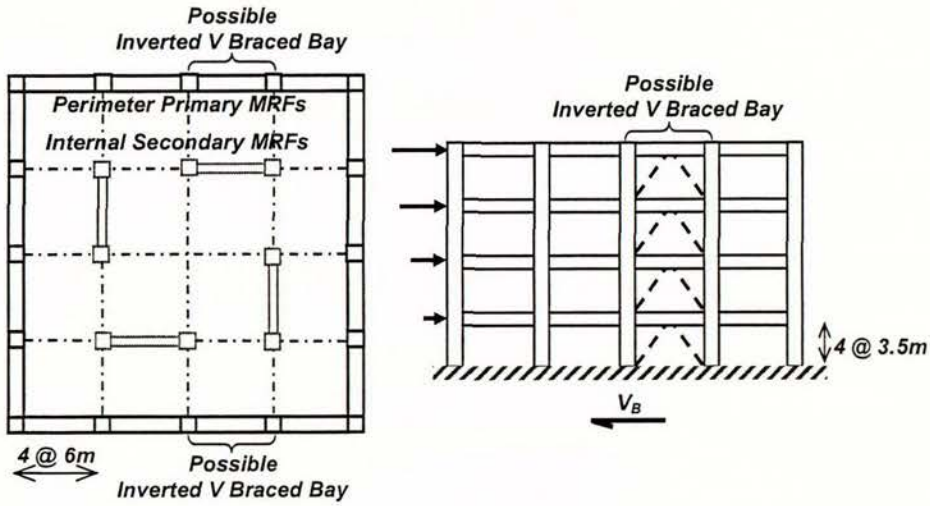


Figure 1: Plan and elevation of study building showing primary moment-resisting frames (MRF) and internal secondary MRF positions. Also shown are the bays used for the buckling restrained brace frames (BRBF) with inverted V form.

3. EFFECT OF INCREASING POST-YIELD STIFFNESS ON RESIDUAL DEFORMATIONS

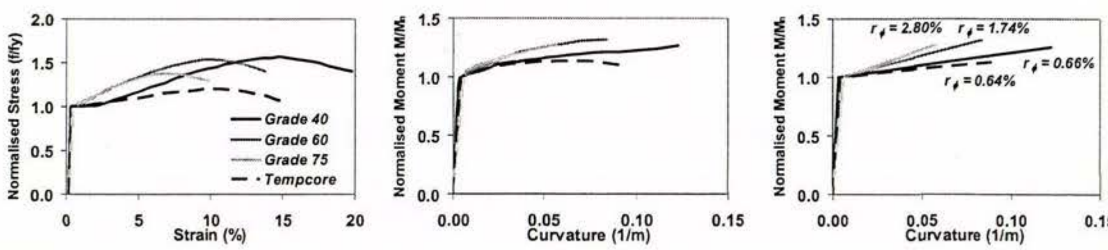
Previous studies [MacRae and Kawashima, 1997; Kawashima et al., 1998; Borzi et al., 2001; Pampanin et al., 2002] have shown that the post-yield stiffness to initial stiffness ratio is the principal factor governing the residual deformation response of a structure. Physically P-Δ is the primary influence on the post-yield stiffness ratio, for which it has been shown in these previous studies that residual deformations are much more sensitive to P-Δ than maximum deformations. In particular it appears that systems exhibiting a post-yield stiffness ratio (on development of a full lateral mechanism) greater that 5% will have significantly reduced permanent displacements. Therefore if simple alterations [MacRae and Kawashima, 1997; Christopoulos and Pampanin, 2004] can be made to a system and its components such that the post-yield ratio is close to or above 5% it could be expected to attain a higher performance level with respect to residual deformations, without significantly altering the maximum response.

4. METHODS TO DIRECTLY ALTER PRIMARY SEISMIC-RESISTING SYSTEM BEHAVIOUR

4.1 Effect of Material Properties on Post-Yield Stiffness

The first consideration comes at a material level. For reinforced concrete both the effect of changes in the concrete and reinforcing steel can be investigated. Using moment-curvature analyses on typical sections with constant steel ratios and section dimensions, the differences in post-yield behaviour can be defined. In this study the influence of concrete compression strength and confinement, and reinforcing steel stress-strain behaviour was considered. It should be noted that in all cases the results for the sections considered were normalised with respect to the nominal yield moment such that comparisons were made only of the moment-curvature curve shape. It is recognised that this does not necessarily represent the exact options available to a design engineer, but is carried out in order to highlight that given new or varying alternatives (of material behaviour) certain aspects of section response can be significantly altered.

Preliminary moment-curvature analyses showed that concrete compression strength (with results normalised by nominal yield moment) has no noticeable effect on the post-yield behaviour of a section. Increased confinement of a section produced minor but generally insignificant changes, therefore these two factors can be discounted. The influence of the reinforcing steel stress-strain behaviour was significant. Four types of steel commonly available in the northern hemisphere were considered; from North America Grade 40, 60 and 75, while from Europe a stress-strain curve representative of Tempcore steel was included. The comparative normalised (f/f_y) stress-strain curves used for each bar type are shown in Figure 2a.



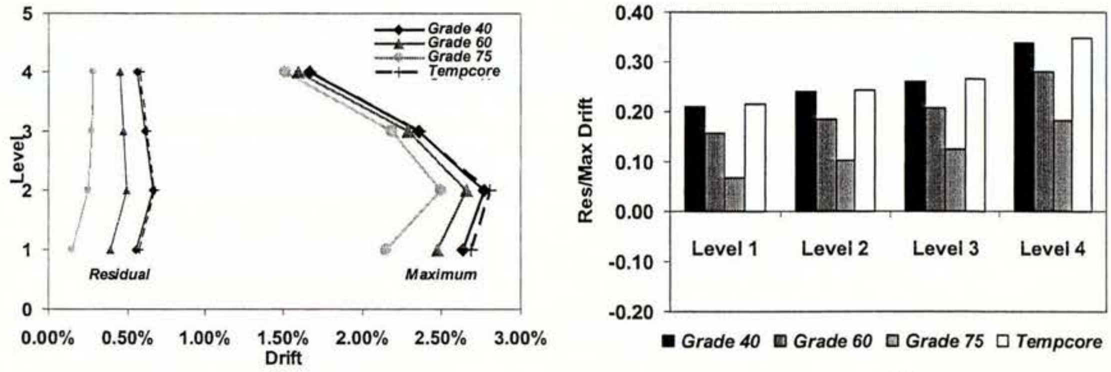
(a) (b) (c)

Figure 2: Normalised (a) reinforcing steel stress-strain curves (b) full M-ϕ response (c) bilinear approximation to M-ϕ response

The resulting normalised (M/M_n) moment-curvature plots for a 400x750 section with equal top and bottom longitudinal reinforcing (total steel ratio ρ_s = 2.4%) and typical transverse reinforcing (ρ_l = 0.25%) are shown in Figure 2b. The bilinear approximation of these curves are shown in Figure 2c where the nominal yield moment is calculated according to concrete and steel strain limits suggested by Paulay and Priestley [1992].

It is clearly seen in both Figure 2b & c that the different reinforcing types give markedly different section post-yield stiffness values. While the ductility of the section tends to decrease with increasing yield stress, it can be expected that performance-based design procedures would not require sections to reach such significant curvatures. From Figure 2c it is seen that for typical reinforcing (i.e. Grade 60) the post-yield ratio is around 1.5 – 2.0% as commonly assumed in computer hysteretic models. In comparison Grade 75 steel produces a post-yield ratio 60% higher than this value and therefore can be expected to produce lower residual deformations. To test this hypothesis a series of inelastic time-history analyses were carried out using lumped plasticity modelling (Carr, 2005).

Figure 3a shows the maximum and residual profiles for the reinforced concrete frame subject to the Caleta 1985 record at an intensity of 150% (the intensity was set 50% higher in order to ensure significant ductile development and storey drifts around 2.5%). It is evident that the maximum drifts attained are slightly reduced (5-15%) by the change in post-yield stiffness, however the residual drifts are reduced by around 33% when comparing the Grade 60 and 75 results, a point clearly shown in Figure 3b by the residual/maximum drift values. Because of the natural re-centring tendency of reinforced concrete, such a reduction in residual response could well be sufficient to raise the performance level of the building, such that it meets code defined performance levels.



(a) (b)

Figure 3: Four storey frame response under Caleta 1985 record (a) inelastic time-history maximum and residual drift values (b) comparative residual/maximum drift ratio values

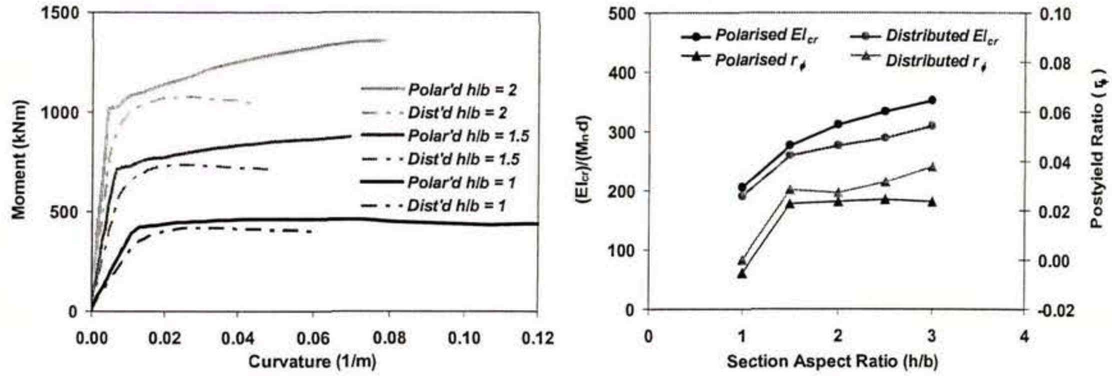
Cleary similar results could be expected for structural steel frames. Such a comparison could be drawn between typical rolled open-sections and cold-rolled tubular sections, the latter having a significant level of strain hardening and more rounded yield curve due to the residual stresses present with such sections.

4.2 Effect of Section Design on Post-Yield Stiffness

Vertically distributing the longitudinal steel in a beam section rather than lumping the bars at the top and bottom of the section, as traditionally assumed by designers, can have a positive influence on residual deformation behaviour. The option of using 'distributed' steel instead of 'polarised' (lumped) steel was experimentally investigated by Wong et al. [1990] as part of a proposed approach to reduce the amount of joint core reinforcement. The sections with distributed steel were found to attain similar flexural capacities as the traditional polarised sections. From the perspective of residual deformations, sections with distributed steel exhibit a more gradual yield and capacity curve, as well as lower unloading/reloading stiffness. This compares to a similar section with polarised steel, which will generally have a very well defined yield curvature and maintains a higher unloading/reloading stiffness.

It should be noted that polarised steel layouts will force the longitudinal tension steel into greater peak strains, thereby developing larger post-yield stresses, and higher apparent residual stiffness. It is however expected that cyclically the distributed steel sections would demonstrate lower residual drifts because the softer unloading and reloading yield behaviour will influence more of the nonlinear response, rather than the peak strains of polarised sections which will be reached over a limited number of cycles. The response for a range of aspect ratios is shown in Figure 4.

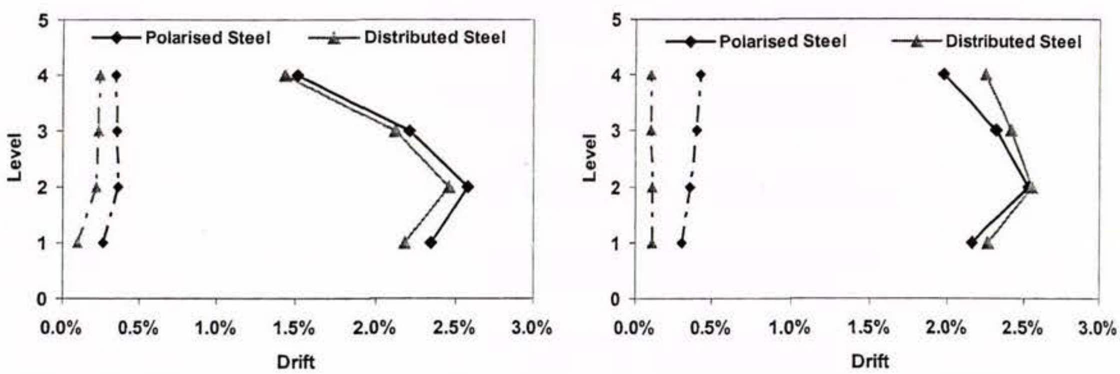
Comparing the actual M-ϕ curves it is apparent that nominal moment capacities are approximately equal between sections with polarised and distributed steel, however the distributed sections develop significantly lower overstrength moments due to lower steel strain hardening, a useful side-effect for capacity design considerations, also noted by Wong et al. [1990]. The softer yield curves of the distributed steel sections are also clearly evident (Figure 4a). In Figure 4b the comparison between flexural cracked stiffness and post-yield ratio is shown for varying aspect ratios. In all cases the distributed sections have a lower cracked stiffness, however they have a consistently higher post-yield stiffness ratio (as defined from the standard bilinearisation approach described earlier). Note that Figure 4b highlights the slight increases in post-yield ratio with aspect ratio (and decreasing longitudinal steel ratio) due to the higher steel stresses developed.



(a) (b)

Figure 4: (a) M-ϕ response of polarised (Polar'd) and distributed (Dist'd) sections with varying depth to width aspect ratio (h/b) and equal flexural steel area (b) comparison of normalised cracked stiffness and post-yield ratio for polarised and distributed steel layouts.

The maximum and residual drift profiles in Figure 5 clearly show that the vertically distributed steel layout reduces the residual storey drifts while influencing the peak drifts to a lesser extent. The actual development of the differing residual drifts is shown by the time-history in Figure 6, where it is evident that peak behaviour is not greatly affected, but that following the peak response the distributed steel sections tend to re-centre during the low amplitude cycles. The effect is more evident in Figure 5b for the fibre-element modelling which allows the explicit definition of the reinforcing layout with each beam section and therefore better captures the full non-linear response and reductions in residuals due to both post-yield stiffness and lower unloading/reloading stiffness. The differences in peak drifts between Figure 5a & b are attributed to slight differences in modal damping and the use of the cracked initial stiffness for the lumped plasticity approach compared to the uncracked stiffness of the fibre-element sections (no allowance is given for cracked initial stiffness in SeismoStruct). The comparative behaviour of the two different modelling approaches highlights some particular issues which should be considered when using inelastic time-history analyses for design verifications.



(a) (b)

Figure 5: Four storey frame response under Caleta 1985 record: comparing response of polarised and distributed reinforcing layouts (a) using lumped plasticity model (b) using fibre-element model.

5. INTRODUCTION AND DESIGN OF SECONDARY ELASTIC SEISMIC-RESISTING SYSTEMS

In previous studies on residual deformations the inherent hysteretic differences between typical well detailed reinforced concrete and structural steel behaviour have been found to significantly influence the magnitude of the final displacements with respect to the corresponding maximum displacement. The degrading stiffness of reinforced concrete sections tends to cause a natural re-centring of the element under small amplitude cycles, whereas structural steel does not exhibit such significant stiffness degradation, thereby tending to maintain larger residual displacements. The implication is that well designed reinforced concrete structures are not as susceptible to residual deformations as similar structural steel buildings. However the flexibility of material and section level details available to the designer in reinforced concrete are not necessarily applicable to structural steel design. What remains are considerations at a system level and the possibility to achieve sufficiently high system post-yield stiffness that global residual deformations will be acceptable.

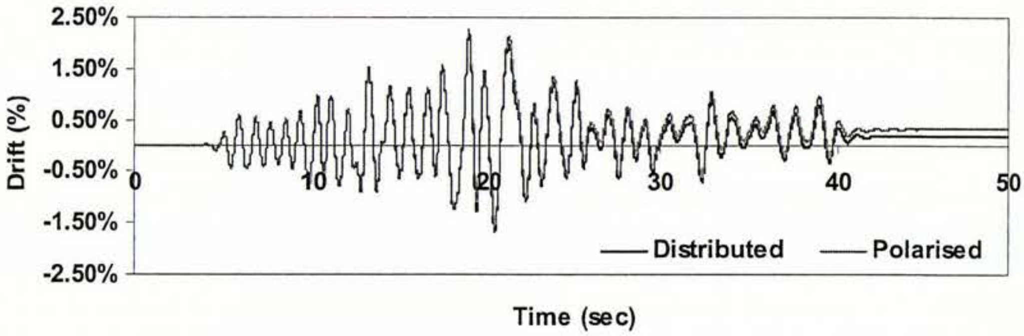


Figure 6: Time-history of 4-storey RC MRF at effective height (at the centre-of-force) comparing polarised and distributed beam reinforcement layouts (using Ruaumoko 2D).

A particular form of seismic resisting system, the Buckling-Restrained Braced Frame (BRBF) [Watanabe et al., 1988], has been shown to be vulnerable to concentrated residual deformations due to the very stable, larger hysteretic loops generated from the pure tension-compression yield within the brace members. A recent study by Kiggins and Uang [2006] demonstrated that the residual deformations of BRB frames could be appreciably reduced by the inclusion of a secondary resisting system, in this case an internal gravity MRF. Clearly this concept can be extended to other primary structures, both steel and reinforced concrete that may exhibit significant residual deformations.

The inclusion of a secondary seismic resisting system to assist a primary inelastic resisting system is a simple concept, and with proper detailing easily achieved. While such a system could be additional to the structure already present within an original plan, it is more efficient to look at the contribution made by other elements or frames that would be present within the building. The secondary MRF considered in addition to the BRBF bay by Kiggins and Uang [2006] was stated as being an internal gravity frame already present within the plan of the structure, however it was assigned 25% of the design base shear of the original braced bay.

5.1 Problem Definition and Design Strategy

The intent of including or activating a secondary system in the seismic response is to increase the global post-yield stiffness of a structure such that the residual drifts are reduced to an acceptable level. By assigning an arbitrary 25% of the design base shear to the secondary frames, Kiggins and Uang [2006] did not explicitly consider the global post-yield stiffness ratio in the design of the secondary system. However if a primary resisting system, be it MRF, braced-frame or flexural wall is considered to act in parallel with a secondary system that remains elastic, the global force-displacement post-yield stiffness can be described as:

$$K_{system,postyield} = r_{\Delta,1} \cdot K_{1,elastic} + K_{2,elastic} \geq r_{\Delta,target}\left(K_{1,elastic} + K_{2,elastic}\right) \tag{1}$$

The inclusion of the inequality implies that a global post-yield stiffness ratio is sought, greater than or equal to $r_{\Delta target}$ which is a function of the primary system elastic stiffness $K_{1,elastic}$, the primary post-yield stiffness ratio $r_{\Delta,1}$, and the secondary system elastic stiffness $K_{2,elastic}$, all of which must account for the reductions due to P-Δ effects as described in a general form by MacRae [1994]:

$$K_p = K_o(1-\theta) \tag{2}$$

where K_p is the P-Δ modified stiffness, K_o is the stiffness without P-Δ and θ is the stability ratio equal to:

$$\theta = \frac{P}{K_o h} \tag{3}$$

with P the vertical destabilising load and h the storey or effective height under consideration. Finally the adjusted post-yield stiffness ratio is defined as:

$$r_{\Delta p} = \frac{r_{\Delta o} - 1}{1 - \theta} \tag{4}$$

where $r_{\Delta o}$ is the system post-yield ratio without P-Δ effects.

Considering the SDOF analyses and resulting Residual/Maximum drift design spectra presented by Christopoulos et al. [2004] it becomes apparent that $r_{\Delta target}$ could be set as low as 5% to 10%, above which point residual deformations are not significantly reduced further. A conservative simplification can be made if it is assumed that the value of $r_{\Delta,1}$ is negligible (but not significantly negative) and can therefore be ignored. Thus the only source of post-yield stiffness for a system at maximum response is $K_{2,elastic}$. It becomes clear that by defining closed form solutions for each stiffness contribution it is possible to explicitly solve for the secondary system member properties such that the inequality of Eq.(1) is satisfied.

5.2 Explicit Solution for Secondary System Definition

The solutions presented here will focus on structural steel design, however the same approach can be applied for reinforced concrete systems although definition of each section moment of inertia will need some consideration due to the interdependence of strength and stiffness. Englekirk [1994] presents a series of closed form stiffness solutions for typical steel structural forms, including multi-bay MRF and single-bay braced frames. In developing these secondary system solutions it should be kept in mind that ideally the minor system will not yield, or if it does so, that plastic deformations are not significant and do not occur in places that will negate the target effect. To this extent it is preferable that moment-resisting secondary frames do not develop plastic hinges at the column bases (i.e. they are pin-based). Similarly primary braced-frames only suffer yield in the brace members themselves (not considering eccentrically braced frames) and can be assumed to carry axial loads only.

Having decided on the preferred form of secondary system, the total design base shear is determined as usual (by either force-based or displacement-based design methods), however the apportioning of system strength is defined based on $r_{\Delta target}$. Assuming displacement compatibility between the two systems the proportion taken by the secondary system is given by:

$$V_2 = \frac{V_B\left(r_{\Delta,T} - r_{\Delta,1}\right)}{1 - r_{\Delta,1}} \tag{5}$$

With V_1 and V_2 defined, the primary system is designed such that the section properties are available as input to the following secondary system design equations. It should be noted that for reasonable values of $r_{\Delta target}$ the proportion of base shear attributed to the secondary system is likely to be in the range of 5-10%, a value significantly less than the 25% used by Kiggins and Uang [2006]. Note however that such a small reduction in base shear carried by the primary system will, in the case of steel construction, mean that section reductions are often not possible due to the limited range of sizes available.

5.2.1 Primary BRBF with Secondary Pin-base MRF

The most likely form of secondary system is an internal gravity MRF. For a primary inverted V-braced frame (as typically used for BRBF) and secondary pin-based MRF, the following solution can be defined based on the equations from Englekirk [1994]:

$$K_{1,elastic} = \frac{A_{c1} E L_1^2 A_{d1}}{4 L_d^3 A_{c1} + A_d L_1 h^3} \tag{6}$$

where A_{c1} and A_{d1} are the areas of the primary columns and braces respectively, L_1 is the braced bay length and L_d is the brace length.

The secondary system stiffness, ignoring flexural contributions, is defined as:

$$K_{2,elastic} = \frac{6E}{h^2}\left(\frac{2I_{b2}}{L_2} + \frac{I_{c2}}{h}\right) \tag{7}$$

where I_{b2} and I_{c2} are the beam and column second moments of inertia, h is the storey height and L_2 is the secondary bay length. Substituting Eq.(6) and Eq.(7) into Eq.(1) and assuming that I_{b2} equals I_{c2} the following solution can be found:

$$I_{b2} = \left(\frac{r_{\Delta,target}}{1 - r_{\Delta,target}}\right)\left(\frac{A_{c1}L_1^2 A_{d1}}{4L_d^3 A_c + A_d L_1 h^3}\right)\left(\frac{h^3 L_2^2}{12h + 6L_2^2}\right) \tag{8}$$

Equation (8) is defined for each storey, however for regular structures and design efficiency it is appropriate to design for the bottom floor response and apply the chosen sections over a number of storeys (as generally carried out for primary system design). Note that a further check can be made on the secondary system members to determine if they are likely to yield under design drift limits using the simplified yield drift equation for frames proposed by Priestley [1998] and defined as:

$$\theta_y = \Lambda \varepsilon_y \frac{L_2}{h_b} \tag{9}$$

with L_2 as above, h_b the beam depth, ε_y the yield strain and the constant Λ equal to 0.5 for reinforced concrete and 0.6 for structural steel. This equation can then be used to calculate storey yield displacements, which in the case of the elastic secondary system must be greater than the design displacement (Δ_D) or storey displacement corresponding to a code drift limit. Therefore Eq.(9) can be redefined as:

$$h_b = \Lambda \varepsilon_y \frac{L_2}{\Delta_D} h_i \tag{10}$$

where h_i is the storey height under consideration. Equation (10) is not exact, therefore provided that the section depth defined by Eq.(8) is close to h_b it can be expected that the secondary beam members will maintain close to elastic response.

For the values of $r_{\Delta target}$ suggested above it is unlikely that strength capacity will be critical, however an elastic analysis should be completed on the secondary system to ensure that beam and column member sections defined from Eq.(8) are sufficient to carry the proportion of base shear defined by Eq.(5). If necessary column sections for secondary frames can be designed to meet capacity design requirements although provided the primary system is capacity designed it should control the inelastic response over the height of the structure.

5.3 Implementation of the Design Procedure for Secondary Systems

To test the proposed procedure the BRBF mentioned previously was redesigned for a system post-yield ratio, $r_{\Delta target}$, equal to 5%. The brace-core area was reduced to account for the lower design forces as such sections are generally fabricated specifically. The resulting BRBF dimensions and strengths along with secondary MRF sections are described in Table 1. Gravity load dominated the secondary section demands, therefore they were accordingly sized for gravity load capacity. Note that the BRBF braces were modelled with an effective area representing the contribution to elastic stiffness from the complete brace construction [Tremblay et al., 2004].

Both the original primary frame and dual system were subjected to the Caleta 1985 earthquake record at 150% of the design intensity in order to ensure significant ductility development. The resulting time-history is shown in

Figure 7. It is clear that the addition of a secondary MRF does not alter the shape of the drift or period response of the structure, but does limit the cumulative unidirectional build-up of lateral drift in the inelastic cycles. It should be noted that the reduction in residual drift is similar to the average reduction found by Kiggins and Uang [2006], indicating that additional strength allocation to the secondary system (or higher values of $r_{\Delta target}$) is unlikely to produced markedly different results. This reflects the findings of Christopoulos and Pampanin [2004] which suggested that increases of post-yield stiffness ratio above 5% do not significantly reduce residual deformations further.

Table 1: Section details for 4-storey steel primary BRBF and secondary MRF

	Primary BRBF $V_B = 2465$kN	$V_1 = 2335$kN			Secondary MRF $V_2 = 130$kN	
Level	Brace Effective Area (mm^2)		Core Tensile Strength (kN)		Columns	Beams
	Original	With Secondary	Original	With Secondary		
4	2956	2799	581	550	250UB37	250UB37
3	6203	5874	1219	1156	250UB37	250UB37
2	8403	7958	1651	1565	250UB37	250UB37
1	9379	8882	1843	1747	250UB37	250UB37

V_B = Total base shear; V_1 = Primary frame base shear; V_2 = Secondary frame base shear;

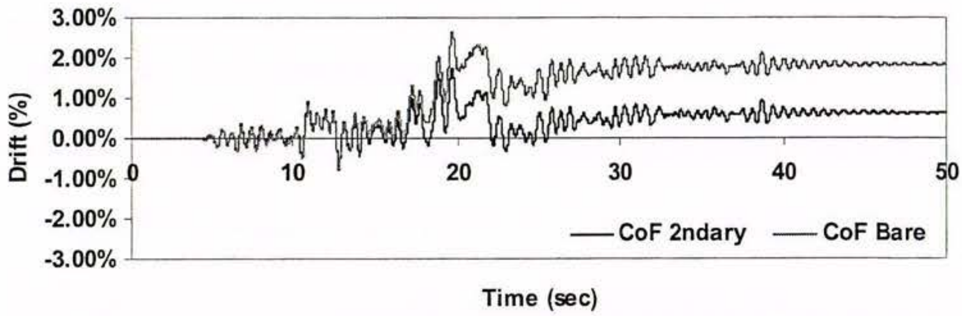


Figure 7: Time-history response at the centre-of-force (CoF) of the initial 'Bare' BRBF with no secondary frame and the comparative response when a secondary frame is included.

5.4 Comments on the Inclusion of Secondary Systems in Prototype Structures

As described, the most obvious form of secondary system is an internal gravity frame that would already be present within a proposed structural plan. Thus in many cases it may not be necessary to provide an additional frame or wall, but simply to redefine the role of certain elements such that they are satisfactorily contributing to the seismic resistance of the structure. To this extent the designer must ensure that floor diaphragms adequately connect the primary and secondary lateral load resisting systems. Given the relatively small amount of additional elastic stiffness required to raise the post-yield stiffness ratio above 5% it is plausible that with certain floor slab seating forms the out-of-plane stiffness of orthogonal seismic frames or walls may be mobilised, and be sufficient to provide the required amount of elastic contribution.

6. CONCLUSIONS

The ability to estimate and account for residual deformations in structures implies that design engineers should then be able to apply changes to the structural design such that the likely permanent displacements are effectively reduced or mitigated. Previous studies have identified the post-yield stiffness ratio as the critical influence on residual deformation behaviour. A series of simple approaches aimed at increasing the member and/or system post-yield stiffness ratio in both reinforced concrete and structural steel design have been presented and demonstrated using moment-curvature and inelastic dynamic analyses on simple code-compliant structures. These methods consider changes available to the designer at material, section and system levels, with increased strain hardening levels, vertically distributed beam flexural steel and secondary elastic seismic systems being exemplified as particularly effective at achieving increases in post-yield ratio.

ACKNOWLEDGEMENTS
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MITIGATION STRATEGIES AND SOLUTIONS

TASK 3 – Implementation of self-centering systems

Frame Buildings

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Bridges

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First European Conference on Earthquake Engineering and Seismology
(a joint event of the 13th ECEE & 30th General Assembly of the ESC)
Geneva, Switzerland, 3-8 September 2006
Paper Number: 794

QUASI-STATIC AND PSEUDO-DYNAMIC TESTING OF DAMAGE RESISTANT BRIDGE PIERS WITH HYBRID CONNECTIONS

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SUMMARY

An increasing interest in the development of high-performance seismic resisting systems based on jointed ductile connections, comprising of unbonded post-tensioning techniques, has been observed in the past decade. An extension of this technology, originally proposed for precast building systems, to bridge pier and systems has been recently proposed as a viable and promising alternative to traditional monolithic cast-in-place construction. In particular, specific interest has been given to the efficiency of the 'hybrid' solution which provides a type of 'controlled rocking' at the critical section. Unbonded post-tensioned tendons are combined with an appropriate proportion of mild steel energy dissipation, limiting deformations to a single gap opening and minimising damage and residual deformations when compared to equivalent cast-in-place solutions.

As part of a comprehensive research program at the University of Canterbury, a series of quasi-static and pseudo-dynamic tests on 1/3 scale cantilever bridge piers in either a post-tensioning only or a hybrid configuration, have been carried out. Both internal and external dissipation devices (tension-compression yielding) have been adopted for the hybrid solutions. Results are presented and compared with the performance of an equivalent monolithic cast-in-place specimen used as a benchmark specimen. Confirmation of the expected high-performance of the hybrid systems are given when compared to the response of the equivalent monolithic specimen: a significant, low level of physical damage as well as negligible residual displacements are in fact observed. Further validations and refinements of simple lumped plasticity modelling approaches are also presented and discussed.

1. EVOLUTION OF CONTROLLED ROCKING BRIDGE PIER CONNECTIONS

Over the past decade a significant amount of damage has been observed to a number of bridges and buildings, often beyond the repairable limit. This has resulted in further pressure being imposed upon engineers and designers to provide societies with structures which minimise structural damage, repair costs and downtime after a seismic event. As a consequence, major efforts have been undertaken to develop innovative concepts in seismic resistant systems able to limit damage and related costs.

The progress of the U.S. PRESSS research project coordinated at the University of San Diego (Priestley et al. 1999), introduced innovative jointed ductile post-tensioning systems for use within seismic resisting systems. A particularly efficient solution, referred to as a "hybrid" system, relying on the combination of re-centring properties of unbonded post-tensioning tendons in conjunction with dissipative internal mild steel elements, have been extensively developed. Under seismic excitation, the system develops a type of controlled rocking upon the critical section (column-foundation, wall-foundation, beam-column connection). Inelastic deformation is

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concentrated at a single gap opening, directly activating the dissipation characteristics of the mild steel reinforcement and tendon elongation providing an inherent self-centring contribution, thus limiting residual deformations and damage to structural systems.

Figure 1a illustrates the extension of the hybrid system to a bridge pier, having the critical interface located at the pier-foundation connection. Figure 1b highlights the cyclic behaviour of the hybrid system, in terms of energy dissipation and residual deformations, varying the relative ratios of $M_{pt}+M_N$ (non-linear elastic behaviour provided by the unbonded tendons and axial loads) and M_s (elasto-plastic or similar behaviour given by mild steel or energy dissipation devices), thus defining the parameter $\lambda=(M_{pt}+M_N)/M_s$ (Palermo et al. 2005a), (NZS3101:2006).

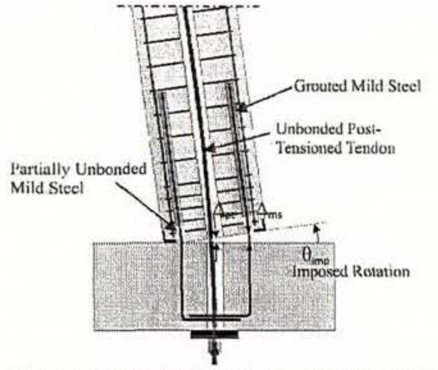


Figure 1a: Hybrid bridge pier schematic

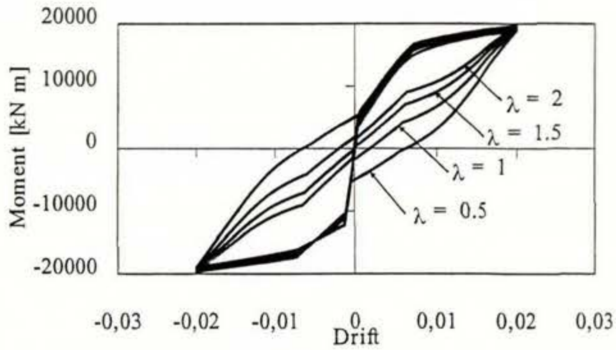


Figure 1b: Dissipation characteristics of hybrid systems

The λ parameter becomes essential especially when considering a displacement based design approach (Priestley 2002), where an estimation of the equivalent viscous damping is required within targeting specific performance limits. Values of λ>2 guarantee limited residual displacements, however maximum displacements may be jeopardised through a reduction in energy dissipation (equivalent viscous damping $\xi_e \approx 12\text{-}15\%$), whereas values ranging from 1 to 1.5 guarantee limited maximum displacements ($\xi_e \approx 15\text{-}20\%$), while providing negligible residual deformations.

2. RESEARCH PROGRAM AND EXPERIMENTAL INVESTAGATION

Three 1/3 scaled hybrid bridge pier systems are experimentally tested using quasi-static and psuedo-dynamic testing regimes. One unbonded post-tensioned solution, and two hybrid solutions (dissipation internal and external to the section profile) are presented. The results of an equivalent monolithic cast in-place bench mark pier are also reported and compared against. Table 1 summarises the testing reported herein, as part of a more comprehensive research program. Additional discussions on previous testing can be found in recent publications (Palermo et al. 2006b), (Palermo et al. 2006a). The actual test matrix is significantly more comprehensive, and for sake of clarity only the immediate testing related to 2D quasi-static and pseudo-dynamic behaviour are herein discussed.

Table 1: Experimental test matrix

Specimen	Expt Name	Initial post tensioning	Dissipation	protocol
Monolithic	**MON_2D**	200kN	16-D10 Grade 300 reinforcing steel (monolithic)	Quasi-static
Post-tension only	**PT1_A**	300kN (4 tendons @ 75kN)	Nill	Quasi-static
	PT1_B	300kN (4 tendons @ 75kN)	Nill	Psuedo-dynamic
Hybrid 300kN Internal	**HBD3**	300kN (4 tendons @ 75kN)	4-HD20 reinforcing bars: 12.5mm fuse diameter @ 75mm length	Quasi-Static
Hybrid 300kN External	**HBD6_QS**	300kN (4 tendons @ 75kN)	4-External dissipaters: 10mm fuse diameter @ 75mm length	Quasi-static
	HBD6_PD	300kN (4 tendons @ 75kN)	4-External dissipaters: 10mm fuse diameter @ 75mm length	Pseudo-dynamic

2.1 Bridge prototype and specimen design

The prototype for the monolithic and hybrid bridge specimen is representative of having a 4.8m high pier with a box girder deck and total participating mass of 180 tonnes (1800kN) for both the solutions. The corresponding 1/3 scaled test specimen consists of a 1.6m high pier with a participating mass of 6.80 tonnes (200kN). The design of the monolithic solution has been carried out using the NZ Concrete Standard Code (NZS3101:2005), for detailing but following a Displacement-Based Design approach (Priestley 2002) (Priestley 2003), with the following performance levels:

1. 2% target drift corresponding to a design level earthquake event (500yr return period)
2. No collapse for maximum credible earthquake (2500yr return period)

Figure 2 indicates the design spectra used for the design of the prototype specimen. It can be assumed that the specific design of the prototype can be related to two different PGA intensity levels and soil conditions according to the NZS1170.5 design spectrum. The design would in fact correspond to either a site PGA of 0.45g located on soil type C (shallow soil) or to a site PGA of 0.28g located on soil type D (deep or soft soil). For comparison with an EC8 standard (EC8 2003), the latter seismic demand would be similar to a site PGA of 0.32g for soil category C (Figure 3). As discussed later, the pseudo-dynamic tests were carried out using two earthquake excitations (Table 2) selected from a suite of 20 earthquakes (Christopoulos and Filiatrault 2002) originally scaled to the UBC 97 spectra (zone 4 soil C or D and PGA 0.4g), which also corresponds to a 2/3 maximum credible event under the ICC spectra, soil category C (Pampanin et al. 2002). The earthquake excitations and the average of their elastic response spectra are shown in Figure 2.

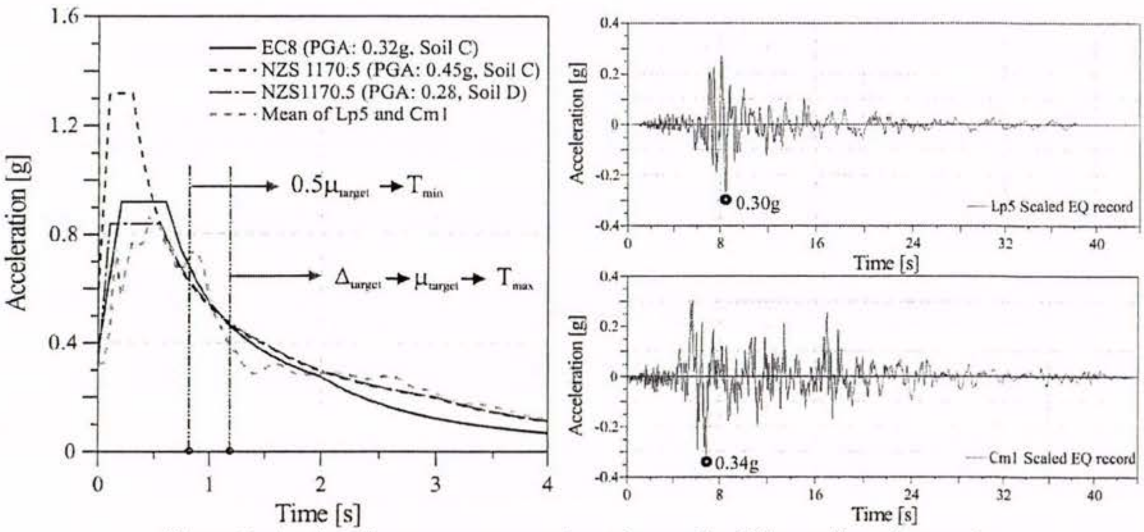


Figure 2: Acceleration response spectra and records of the earthquake events

Table 2: Selected earthquake excitations for pseudo-dynamic testing

Name	Earthquake event	Year	Mw	Station	R_{closed} (km)	Soil type	Scaled PGA
Cm1	Cape Mendocino	1992	7.1	Fortuna Fortuna Blvd	23.6	C	0.339
Lp5	Loma Prieta	1989	6.9	Hollister Diff. Array	25.8	D	0.362

Figure 3: Monolithic construction details, experimental set-up, quasi-static testing protocol, and monolithic pier specimen.

The reinforcement details within the plastic hinge length of the monolithic specimen comprise of 16-D10 (16-10mm diameter bars, grade 300MPa) symmetrically located. Figure 3 illustrates the details of the section, and the experimental test rig used for all pier testing herein presented (for sake of simplicity the specimen shown is representative of the monolithic pier only). The quasi-static testing protocol used for all 2D testing followed the acceptance criteria on pre-cast concrete frame systems proposed by ACI T1.101, ACI T1.1R-01 (ACI_T1.1R-01 2001), is shown in Figure 3. The specific material properties for each of the test specimens are shown in Table 3. Note that PT1and HBD6 have a 25mm steel plate located at the base of the pier, hence concrete compression strength is not as important in terms of the specific deformation behaviour at the base.

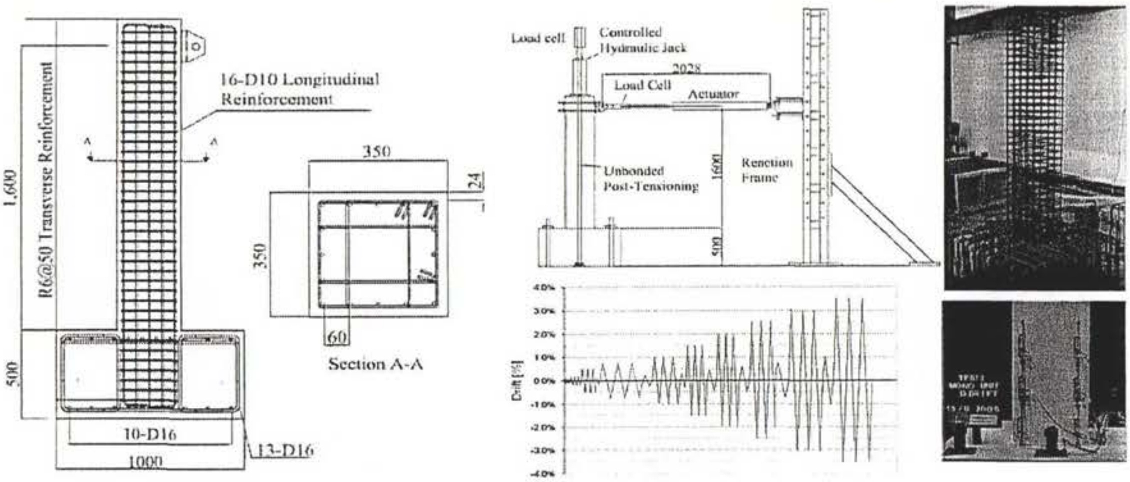


Figure 3: Monolithic construction details, experimental set-up, quasi-static testing protocol, and monolithic pier specimen

The construction details of the two hybrid specimens are illustrated below in Figures 4 and 5. The internal hybrid solution (HBD3, Figure 4) consists of 300kN initial pre-stress and 4-HD20 (4-20mm diameter bars, grade 500MPa), fused to a diameter of 12.5mm over an unbonded length of 50mm. This unbonded length has been properly designed in order to limit the strain demand at the location of the gap opening and prevent premature rupture of the fused bars. The fuse diameter of 12.5mm was designed such that the lateral capacity of the section was comparable to the equivalent monolithic pier. The dissipater attachment is shown in detail in Figure 3, where the fused HD20 bars are threaded at one end, and placed into the foundation via a cast-in-place insert. The pier is then lowered onto the foundation block; having the HD20 bars pass into corrugated ducts, then subsequently grouted in place.

Table 3: Material properties of experimental specimen

Material	Monolithic pier	PT1	HBD3	HBD6
f_c' (7 Day strength)	52.6 MPa	N/A-(40.6MPa)	40.6 MPa	N/A-(40.6MPa)
f_c' (28 Day strength)	66.5 MPa	N/A-(49.5MPa)	49.5 MPa	N/A-(49.5MPa)
f_c' (Day of testing)	65.9 MPa	N/A-(>54.1MPa)	54.1 MPa	N/A-(>54.1MPa)
Steel tensile strength	317 MPa (D10 bar)	-	570 MPa (12.5mm fused, HD20 bar)	330 MPa (10mm fused, mild steel bar)
7-wire pre-stressing strand	-	1600 MPa (yield) 1870 MPa (0.2% proof stress)	1600 MPa (yield) 1870 MPa (0.2% proof stress)	1600 MPa (yield) 1870 MPa (0.2% proof stress)

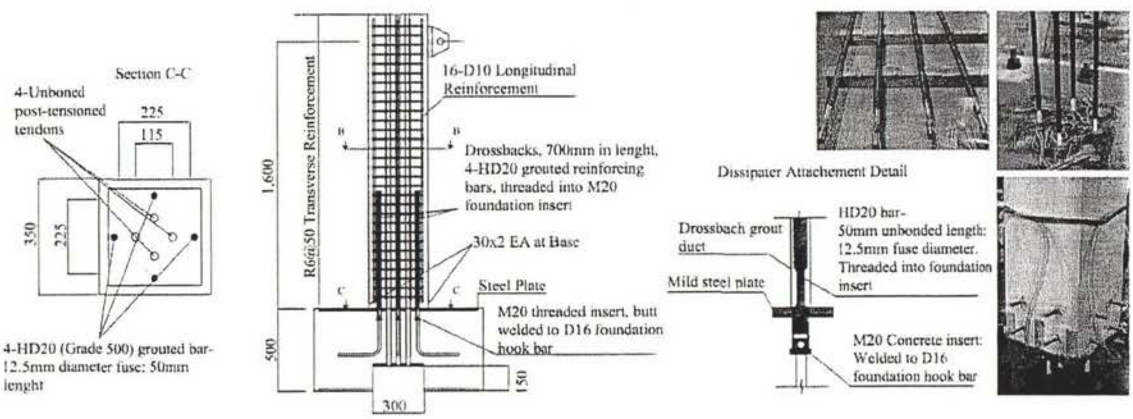


Figure 4: construction details and photos of HBD3 specimen

The external hybrid solution (HBD6, Figure 5) consists of 300kN initial pre-stress and 4-10mm diameter, external steel dissipaters (grade 300MPa). The external dissipaters are made from 20mm mild steel bar, having a fused diameter of 12.5mm over a length of 75mm within a total dissipater length of 300mm. A steel tube acting as an anti-buckling system was placed over the fused length of the dissipater and subsequently injected with either epoxy or grout, in order to guarantee sufficient dissipation of the steel in compression. These replaceable dissipaters are fixed to the pier via steel brackets which are attached to the side of the pier through 4-M25 8.8 high tensile bolts. The bolts supply a torque of 600Nm intended to provide resistance via friction between the steel bracket and concrete pier surface. The resistance was improved substantially by supplying a film of epoxy to the roughed steel bracket and concrete surfaces prior to tensioning of the bolts. The foundation connection is provided through 2-M12 8.8 (per dissipater) high tensile threaded rods having an embedment depth of 250mm, epoxy injected into the foundation block (see Figure 5). It is worth noting that Figure 5 shows steel brackets attached to all four sides of the pier. However, only one direction has so far been tested, reserving the other sides for future bi-directional testing.

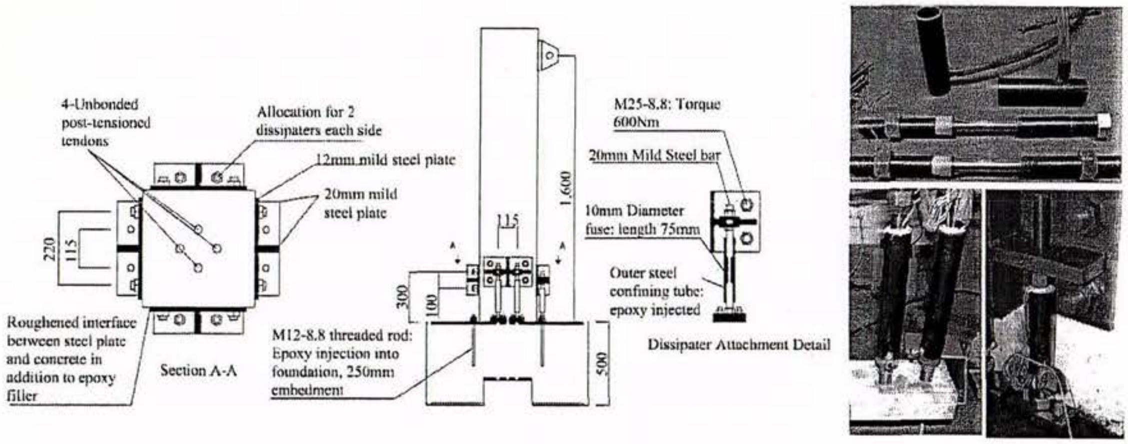


Figure 5: Construction details and photos of HBD6 specimen

The post-tensioned only specimen (PT1) is simply the external hybrid solution without the dissipaters (Figure 5). As the external dissipaters are replaceable, testing could be interchanged between post-tensioning only and external hybrid solutions. It should be recognised that the initial post-tensioning of 300kN for all of the post-tensioning solutions represents 200kN gravity deck loading in addition to a further 100kN provided by unbonded post-tensioned tendons. Practicality issues during the test set-up meant the entire 300kN would be provided by initial pre-stressing of the tendons. Hence in reality, only 1/3 of the axial load upon the prototype would be provided by initial pre-stressing, the remaining 2/3 being provided by the vertical gravity deck loading.

3. MODELING ASPECTS

3.1 Modelling of the Monolithic specimen

Cyclic push pull predictions, herein presented, are based upon lumped plasticity modelling. The finite element code, RUAUMOKO (Carr 2005), is used to implement the model and produce push-pull and dynamic time history responses.

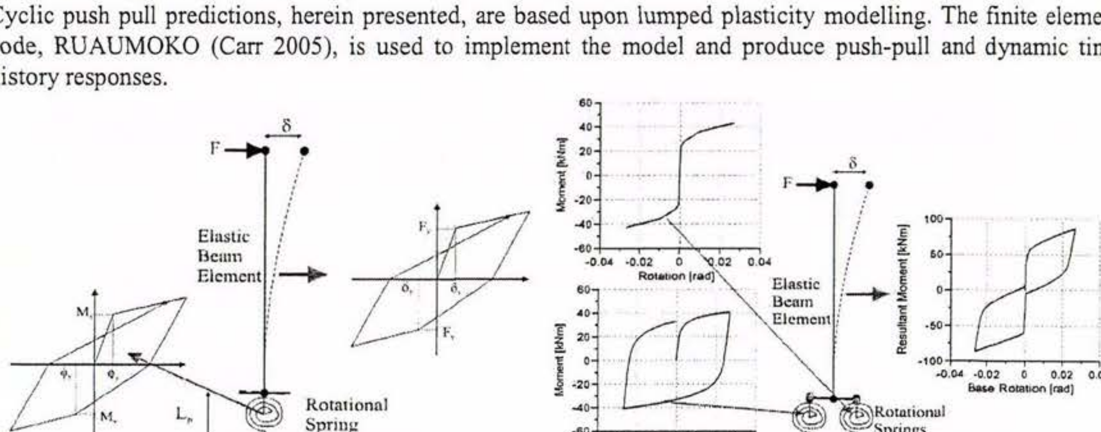


Figure 6: a) Monolithic, lumped plasticity pier model; b) hybrid, lumped plasticity pier model

A modified Takeda hysteresis model is assigned, based upon the back bone curve of the monotonic moment-curvature analysis, where plasticity is lumped within plastic hinge length of the SDOF column element (see Figure 6a).

3.2 Modelling of the Hybrid solution

Traditional moment vs. curvature section analyses are not applicable for hybrid systems. Local steel strain compatibility and Bernoulli's law (plane sections remaining plane) are violated at the critical section as both the mild steel and pre-stressed reinforcement are unbonded within the section. A more appropriate moment vs. rotation section analysis procedure is adopted-based on a global member displacement compatibility condition due to an equivalently reinforced concrete "monolithic" section (referred to as "monolithic beam analogy", MBA), developed by (Pampanin et al. 2001), further refined by (Palermo 2004) which has been extensively adopted after appropriate validation (fib 2003), (NZS3101:2006), (Pampanin et al. 2001), (Palermo et al. 2005b). Inherent within the analytical procedure is the computation of the moment contribution being provided by the unbonded post-tensioned tendons, $M_{pt}+M_N$ (combination of post-tensioned tendons and gravity load), and of the partially unbonded non-prestressed mild steel reinforcement, M_s. Thus, the lumped plasticity modelling of the hybrid pier can be justified through assigning two springs in parallel; one representing the non-linear elastic behaviour of the unbonded tendon and the other emulating the non-linear inelastic behaviour of the non-prestressed mild steel reinforcement or dissipation device (see Figure 6b).

4. EXPERIMENTAL RESULTS AND ANALYTICAL COMPARISONS

4.1 Quasi-static cyclic tests

Push-pull cyclic results of HBD3, having 300kN initial pre-stress and 4-HD20 internal dissipation (fused to 12.5mm), are shown in Figure 7. The analytical prediction (MBA procedure) is overlain the experimental results showing a very good agreement with the theory, further verifying the procedure. Both the yielding point (corresponding to actual yield of the steel) and the geometric non-linearity point (corresponding to a reduction in stiffness due to a sudden relocation of the neutral axis position) are accurately captured. Some stiffness degradation begins to occur at the onset of yield due to bond deterioration of the mild steel reinforcement, however the hysteresis is very stable and the effect is significantly lower when compared to an equivalent monolithic specimen (Figure 10). The hysteresis is characterised by significantly stable energy dissipation, having only minor residual deformations; furthermore the damage sustained to the pier is relatively minimal (Figure 13).

The loads vs. base-rotation behaviour of the two extreme tendons are presented in Figure 7b. The analytical predictions of the tendon load due to elongation are again compared to the experimental results. The analytical procedure slightly overestimates the load in the tendon, however as the actual moment contribution of the tendons is in the order of 60% the total capacity, the difference has little effect to the global behaviour. It should be noted that the load in the tendon at 3.5% drift is significantly lower than the limit of proportionality of the tendon, which is fundamental if residual deformations are to be guaranteed at a target displacement.

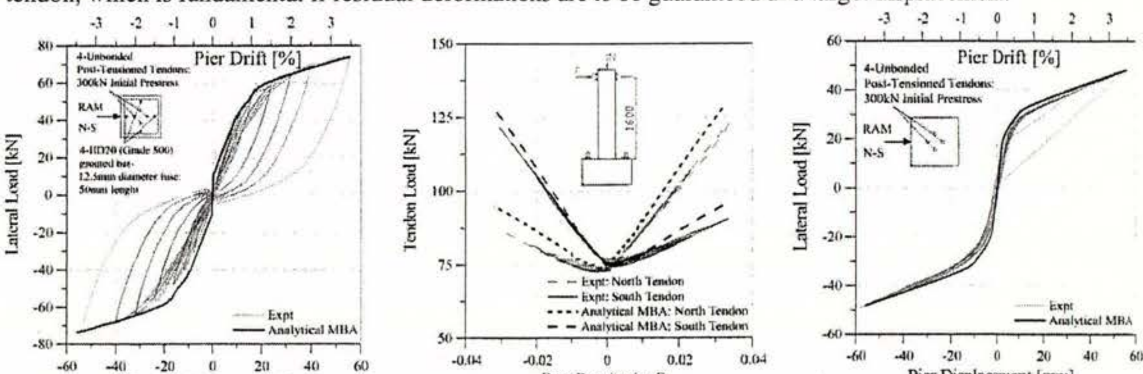


Figure 7: a) HBD3, force-displacement cyclic results; b) HBD3, behaviour of tendon load-rotation; c) PT1, force-displacement cyclic results

The push-pull cyclic response of the post-tensioned only solution (PT1) is shown in Figure 7c. Again it can be seen that the MBA procedure agrees very well with the experimental results. It should be recognised that the predictions for PT1 and HBD6 (hybrid, having external dissipation) are based upon a MBA procedure modified slightly to account for some minor damage to the steel base plate. After repeated testing of the same specimen the perimeter of the steel plate can be expected to undergo some softening/plastic work, thus reducing the initial stiffness of the system.

The experimental and analytical cyclic push-pull and monotonic responses for HBD6 are shown in Figures 8a and 8b. Very stable hysteresis behaviour is observed, in a large part due to the use of external dissipaters as a source of dissipation. The external dissipaters are in fact not subjected to any bond degradation, hence the only stiffness degradation imposed within the system is due to the inherent material softening of the mild steel dissipater due to repeated cyclic loading and minor softening effects coming from localised damage at the base of the pier. This can be seen in Figure 8a below, indicating some stiffness degradation occurring after yield of the pier, however the degradation stabilises. The cyclic behaviour can be compared to the equivalent monolithic pier in Figure 8c, whose behaviour is characterised by significant pinching, however maintaining strength up until a drift of 4.5%. It is evident that, while the monolithic specimen appears to dissipate a significant amount of energy, this is at the peril of substantial physical damage to the specimen, also leading to significant residual displacements. While less energy is dissipated within the hybrid specimen, the cyclic behaviour is significantly more stable and the pier specimen suffers no damage in proximity of what would normally be a plastic hinge zone (see Figure 13).

The monotonic back-bone curve is very accurately predicted through the use of the MBA procedure (modified to account for some damage of the steel plate). Figure 8b illustrates the cyclic behaviour of the analytical model, whose spring properties consist of a tri-linear elastic hysteresis for the unbonded post-tensioned tendon and an Al-Bermani steel hysteresis (available within the Ruaumoko library) for the dissipation component.

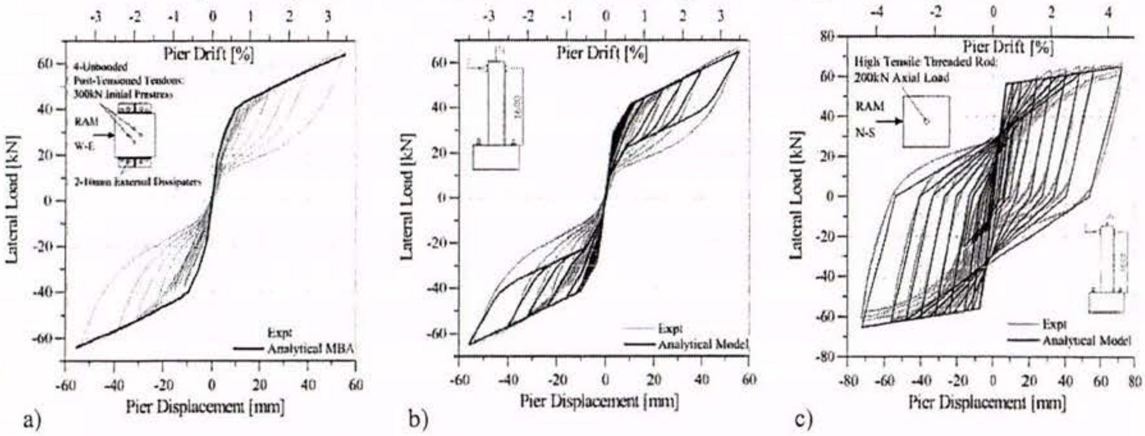
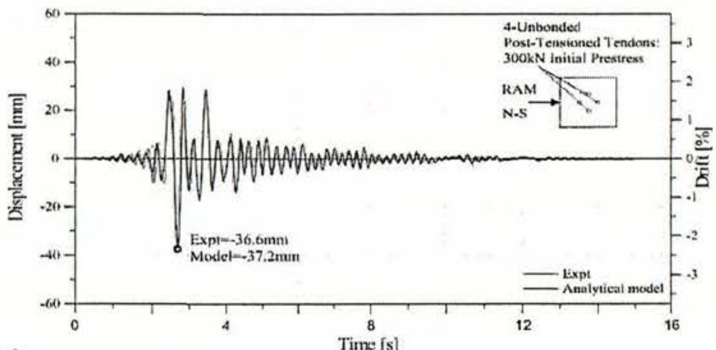


Figure 8: a) HBD6, expterimental force-displacement and analytical MBA envelope; b) HBD6, expterimental and analyitcal cyclic force-displacement; c) expterimental and analytical cyclic force-displacement of equivilent monolithic pier.

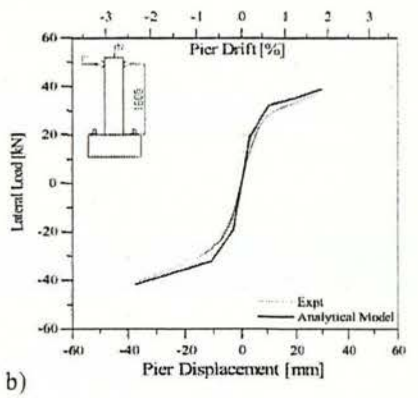
4.2 Pseudo-dynamic tests

The PT1 specimen was then subjected to pseudo-dynamic testing using the aforementioned two earthquake excitations, Lp5 and Cm1 (Table 2 and Figure 2). As mentioned, the records used have been scaled to match the elastic design spectrum used during the design phase of the benchmark monolithic pier specimen. The response of the pier, when subjected to the scaled Lp5 record indicated a likely maximum drift in the order of 2.3% (36.6mm). Figure 9 shows both the experimental test and analytical time history response as generated using the code RUAUMOKO (Carr 2005). The analytical model simply consisted of one rotational spring representing the tendon moment contribution having a tri-linear relationship to best fit the gradual transition into the non-linear range as seen in the quasi-static testing and the MBA analytical response. The model is able to accurately capture the entire displacement time history response in addition to capturing the maximum displacement. Whilst the analytical model slightly over-estimates the initial stiffness and strength of the experimental specimen, the model can still properly predict the global displacement behaviour.

When comparing the experimental-analytical results under the Cm1 earthquake excitation (Figure 10a), a more evident difference is given in terms of the actual-to-predicted displacement time history results. While the overall periodic behaviour is captured, the peaks are not correctly represented by the analytical model. When analysing response spectrum of Cm1 in depth, it can be seen that there is a significant amplification in the acceleration response, and hence the displacement response, beyond a period of approximately 1 second. This roughly corresponds to the "effective period" at onset of the "post-yield" response of the pier, therefore suggesting the displacement response is highly sensitive to the strength of the system (for lightly damped structures, indicative of a post-tensioned only solution). This can be further justified through Figure 10b, which illustrates the strength of the analytical model is slightly greater than the experiment, thus this minor difference in strength would be significant enough to justify the two dissimilar peak displacement response. As a further confirmation, when the strength of the analytical model approaches the strength measured in the experiment, the two displacement responses converge.

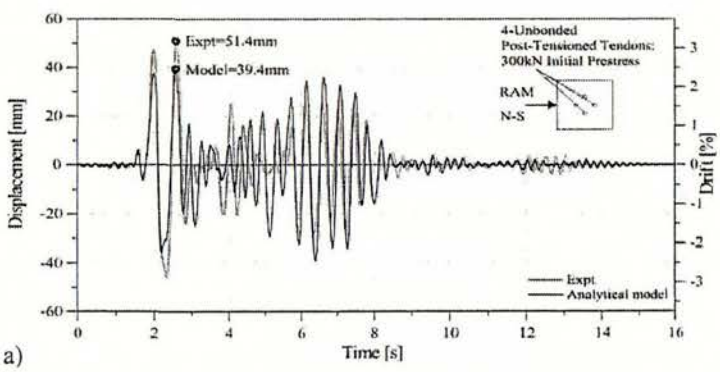


a)

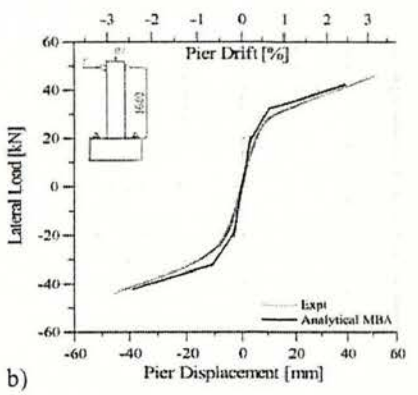


b)

Figure 9a: a) Displacement-time history of PT1 subjected to Loma Prieta (Lp5), experimental and analytical comparisons; b) force-displacement history.



a)



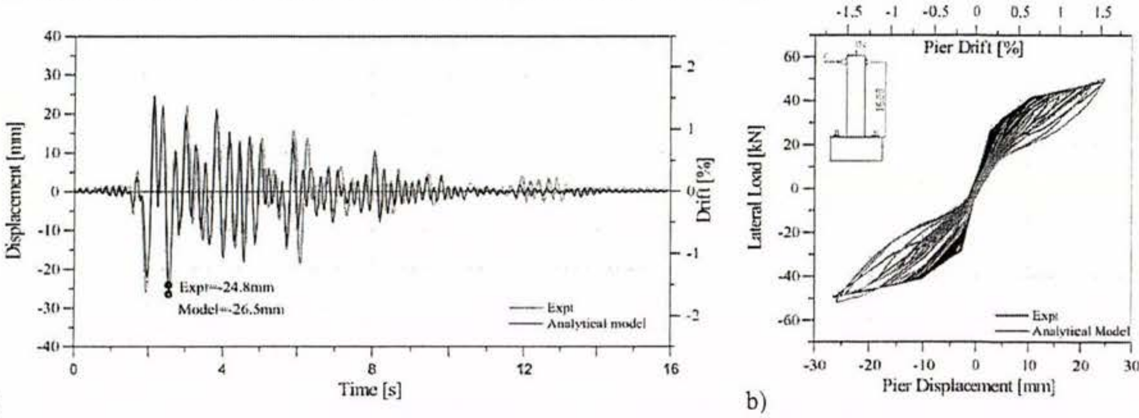
b)

Figure 10: a) Displacement-time history of PT1 subjected to Cape Mendocino (Cm1), experimental and analytical comparisons; b) force-displacement history.

.The displacement history and force-displacement results of the pseudo-dynamic testing of the HBD6 specimen (hybrid solution with external dissipaters), subjected to cml in Figure 11a and 11b, confirm the high performance of the hybrid pier through limited maximum displacements, negligible residual deformations and damage (Figure 13). Furthermore, the targeted maximum drift of 2% for the monolithic pier is not exceeded for the hybrid specimen, thus achieving the required performance criteria-in spite of the lower strength and dissipation capacity. The maximum displacements are comparable between the analytical model and the experimental test, confirming the validity of the simplified modelling approach. It is also evident that the inclusion of damping (hysteretic, in this case) within the model is less sensitive to the response of Cml (when compared with issues previously discussed concerning the post-tensioned only response)-confirmation is evident when the spectra of the Cml record is further analysed, indicating the displacement response is less sensitive for systems with moderate damping.

The two spring model is at this stage, limited in terms of its' capability to accurately capture the entire cyclic force-displacement behaviour to include stiffness degradation and the unloading strength of the real system.

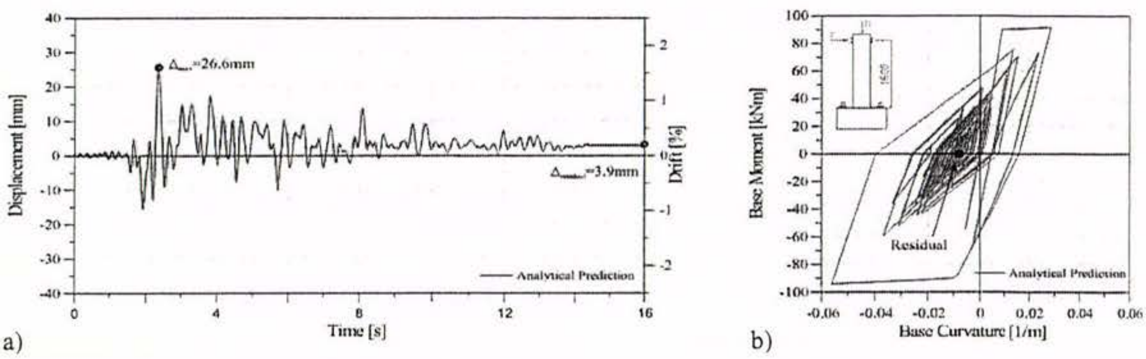
As noted by local testing of the dissipation devices (subjected to cyclic displacements in the positive direction only) the steel elements are affected by some Bauschinger effects only upon unloading in compression following tension elongation. Upon reloading in tension, instead, the Bauschinger effect is not developed due to the device not entering the negative displacement range. More appropriately, two non-symmetric springs with Bauschinger effects acting in one direction only would be required. This is further confirmed when implementing a Dodd-Restrepo hysteresis rule (Dodd and Restrepo-Posado 1995), to model the Bauschinger effects, and capture the entire force-displacement cyclic behaviour. A very reasonable representation is obtained when cycling in one direction only. Alternatively, a multi-axial spring model (whose springs are positioned within the section to locate the actual steel position), implementing tension-only types of hysteretic behaviour would naturally capture such behaviour. Alternative solutions relating to modelling should also be considered for hybrid sections with internally grouted mild steel dissipation elements: in this case, they are not only subjected to Bauschinger effects during the unloading phase, but also suffer considerable stiffness degradation due to bond deterioration about the mild steel element during loading, thus the individual steel element could be properly modelled using a stiffness degradation rule in tension combined with Bauschinger effects for unloading.



a) b)

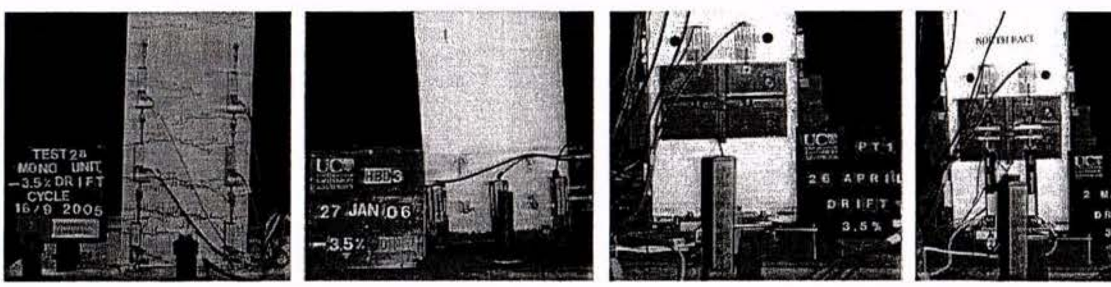
Figure 11: a) Displacement-time history of HBD6 subjected to Cape Mendocino (Cm1), experimental and analytical comparisons; b) force-displacement history.

Furthermore, with respect to Figure 12, the analytical response of the equivalent monolithic pier indicates the displacements of the hybrid solutions are comparable with such a system (in fact, within 0.1mm), moreover the behaviour of the hybrid specimen is significantly more symmetric (more efficient dissipation of energy and damage) and not the subject of any residual deformations. Unfortunately experimental pseudo-dynamic tests on monolithic solutions are still ongoing at the University of Canterbury, preventing an analytical comparison.



a) b)

Figure 12a: a) Displacement-time history of Monolithic subjected to Cape Mendocino (Cm1), analytical model; b) moment-curvature history.



a) b) c) d)

Figure 13: a) Monolithic damage at 3.5% drift; b) HBD3 damage at 3.5% drift; c) PT1 damage at 3.5% drift; d) HBD6 damage at 3.5% drift

5. CONCLUSIONS

The experimental quasi-static and pseudo-dynamic testing on hybrid bridge piers provided encouraging confirmation of an enhanced level of performance when compared to an equivalent cast-in-place monolithic solution. The main advantages of the proposed hybrid systems are the inherent self-centring capability, reducing residual deformations, and the lack of physical damage to the structural elements due to the controlled rocking nature of the jointed connection. Stable energy dissipation along with high levels of ductility and self-centring properties are readily achieved through the proper design combination of advanced elasto-plastic dissipaters and relatively simple post-tensioning construction techniques.

Further verification of the analytical MBA procedure to predict the global force-displacement behaviour of hybrid connections is given, in addition to the simple lumped plasticity modelling approaches used to describe the dynamic response of hybrid bridge pier systems. Based on these preliminary results, further investigations will be carried out on the application of alternative external dissipation devices (elasto-plastic, friction, viscous), in addition to different pier profiles considering also segmental pier systems and various pier-to-deck configurations

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APPLICATION TO RETROFIT

*Pampanin. S., 2006 "Controversial Aspects in Seismic Assessment and Retrofit of Structures in Modern Times: Understanding and Implementing Lessons from Ancient Heritage" *Bulletin of NZ Society of Earthquake Engineering, 39 (2), 120-133, June*

Ireland, M., Pampanin, S., Bull, D.K., Concept and Implementation of a Selective Weakening Approach for the Seismic Retrofit of R.C. Buildings 2006,** ***Proceedings of the Annual NZSEE Conference**, Napier, March**

Concept and Implementation of a Selective Weakening Approach for the Seismic Retrofit of R.C. Buildings

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2006 NZSEE Conference

ABSTRACT: Current seismic retrofit strategies generally focus on increasing the strength/stiffness or upgrading mechanical properties of a structure. A typical drawback with this is that the upgraded behaviour might result in an increased demand on the structural and sub-structural elements, i.e. foundation. Herein proposed is a counter-intuitive but rational seismic retrofit strategy of selectively weakening a structural system. Such a retrofit strategy is suitable for application to alternative seismic resisting systems and components including walls, beams, columns and diaphragm connections.

A selective weakening intervention is performed within an overall performance-based retrofit approach with the aim of improving the inelastic behaviour by first reducing the strength/stiffness of specific members within the structural system. This in turn results in a reduced demand on the structural member. Once weakening has been achieved the designer can use the wide range of techniques and materials available (e.g. use of fibre reinforced polymers, steel plates, jacketing or shotcrete) to ensure that adequate characteristics are achieved. Whilst performing this it has to be assured that the structure meets specific performance criteria and the principles of capacity design.

As the first phase in the development of selective weakening, the feasibility of such a retrofit strategy is discussed, with particular focus on possible applications to the seismic retrofit of existing reinforced concrete structural walls. The proposed intervention involves splitting the wall vertically and cutting it at the foundation level to change the inelastic mechanism from shear-type to a flexural/rocking-type behaviour. As part of the overall research program, a series of experimental (quasi-static cyclic) tests on 2/3 scaled reinforced concrete walls representing pre-1970 construction practice or retrofitted configurations are under preparation. A summary of the retrofit strategy design and expected behaviour will be herein given.

1 INTRODUCTION

Recent earthquake events (e.g. Turkey 1999, 2003 and Taiwan 1999) have highlighted the undesirable behaviour of some existing reinforced concrete structures and the need for appropriate retrofit solutions. Currently two alternative approaches for seismic retrofit are conceptually adopted and implemented in practice: the first approach focuses on reducing earthquake induced forces (i.e. modifying the demand) and the second focuses on upgrading the structure to resist earthquake induced forces (i.e. modifying the capacity, Chuang and Zhuge 2005). In order to reduce earthquake induced forces, base isolation or damping devices are commonly added to the structure, whilst upgrading of the structural capacity is usually achieved by intervening on specific elements or by changing the load paths within the structure. A wide variety of different retrofit techniques for existing reinforced concrete structures including the use of advanced materials (i.e. Fibre Reinforced Polymers) have been extensively investigated and successfully implemented. Issues related to costs, invasiveness and the requirement of specialist knowledge are however typical problems encountered. A comprehensive summary can be found in fib bulletin on seismic retrofit (2003a) and on FRP (2001), while some specific approaches will be mentioned in a later section.

This paper defines and introduces the concepts for an alternative seismic retrofit strategy referred to as a "selective weakening" approach (Pampanin, 2005b) which focuses on protecting undesirable seismic response mechanisms by first strategically weakening specific elements within a structure. Weakening a structure will reduce the seismic demand while at the same time changing the inelastic mechanism according to capacity design principles in order to achieve an overall higher performance level. In a second phase, to achieve a complete retrofit solution other currently available and applicable retrofit techniques can be used in combination with the selective weakening strategy to upgrade the weakened structure to the desired and controlled level of capacity. Recent developments in building technology and high-performance seismic-resisting systems (fib 2003b; Pampanin, 2005a), focusing on the use of a rocking response to ensure minimal damage and to achieve a self-centring behaviour (negligible residual displacements), can for example suggest proper implementation of a selective weakening strategy to existing buildings, whereby the obtained rocking motion is combined with additional dissipation/damping properties for a low level of post-earthquake damage.

The first stage of an ongoing research project which is focusing on investigating/developing selective weakening is underway in the form of experimental and analytical investigations for the application to structural walls. It can be easily anticipated that the selective weakening approach is not limited for the retrofit of structural walls: conceptual applications to frame systems, as well as floor-to-seismic resisting system connection (Jensen et al. 2006) are planned or under investigation as part of a more broad feasibility study of the proposed approach.

2 SEISMIC VULNERABILITY AND RETROFIT OPTIONS FOR EXISTING BUILDINGS

Previous earthquakes have highlighted the poor performance of existing reinforced concrete structures and the need for appropriate retrofit techniques. Figure 1 shows two examples of a shear failure in a reinforced concrete wall due to inadequate transverse reinforcement. Existing buildings may require seismic retrofitting for a number of reasons which include: poor reinforcement detailing, increased loads, revision of design codes, inadequate design philosophy (i.e. lack of capacity design principles).

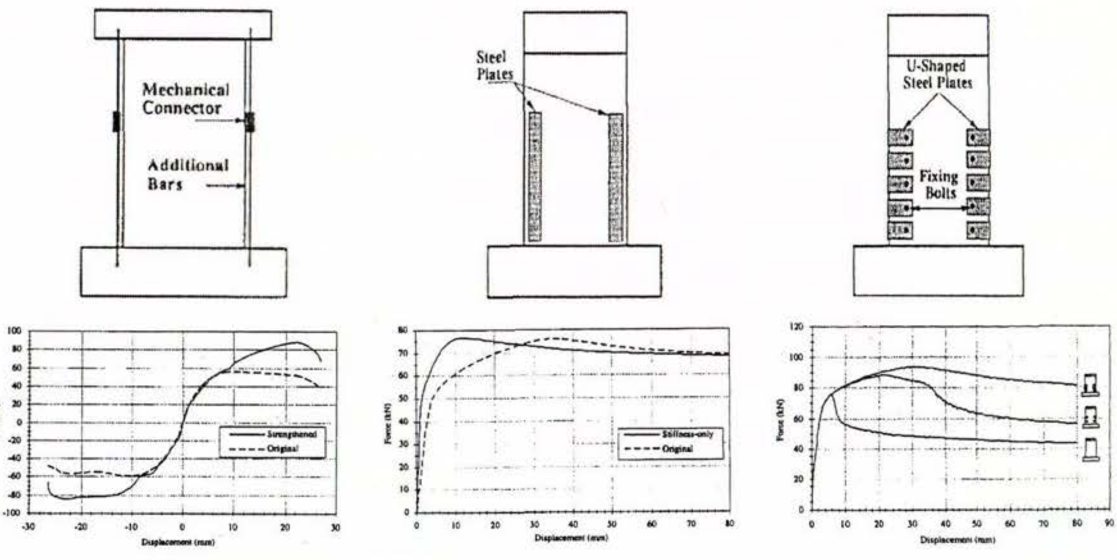
Figure 1: Shear failure of a R.C. due to insufficient transverse reinforcement; a) Bolu (Turkey 1999), b) Bingol (Turkey 2003).

As part of the investigation, a literature review has been carried out to determine typical reinforcement detailing in existing structural walls that require retrofitting to highlight the likely behaviour and to ensure that an appropriate retrofit solution is adopted. It was confirmed that in New Zealand plain round reinforcing bars were used up to the mid 1960s (Liu and Park 2001), and typically a straight lap of about 40 bar diameters was used. It can also be expected that the lap will have insufficient strength to allow the full flexural capacity of the wall to develop and this will cause a bond failure in the lap under seismic excitation. This could be beneficial in saving the wall from significant damage as it will be able to rock, but in turn this could lead to a global failure of the structure. The change to the use of deformed reinforcing bars in the mid 1960s will help increase the capacity of the lap region and as a result the lap is less likely to govern the overall behaviour of the wall. Due to this it is likely that, due to the formation of a plastic hinge, a wall using deformed reinforcement might suffer more damage under seismic response than a wall using plain round reinforcement. On the other hand, the higher strength, stiffness and dissipation capacity developed by using deformed bars should reduce the overall displacement demand of the system.

2.1 Alternative Retrofit Techniques Available

A variety of retrofit strategies for structural walls have been implemented and are available, the most common being concrete jacketing or the use of a shotcrete overlay. These two techniques are conceptually similar, since they involve adding additional reinforcement and a layer of new concrete around the existing wall (a main technological difference being that shotcrete is a form of sprayed concrete (Sabnis et al. 1996)). They result in an effective means for increasing strength, stiffness and ductility but there are several drawbacks, which include: a) need for costly foundation upgrades due to the strength increase; b) higher forces being attracted by the increase in stiffness; c) uncertainty between bond of new and existing concrete; d) labour intensive, time consuming and disruptive type of intervention.

Recent research has been carried out on the development of selective retrofit techniques which aim to offer independent upgrades in strength, stiffness or ductility (Elnashai 1992, Elnashai and Pinho 1998 and Pinho (1999). Selective upgrading offers higher control of the seismic response as the retrofit is directly targeted at upgrading specific characteristics of the wall. The retrofit solutions are largely non complex and consist of placement of steel plates, brackets or external tendons/bars reinforcement. Figure 2 shows a series of examples of selective retrofit techniques and the effects on the force displacement response. Figure 2 (a) show a selective flexural strength upgrade, which is achieved by the addition of external reinforcement, a key aspect to achieve a selective flexural strength upgrade is to ensure that the pre-yield behaviour is not affected. This is achieved by the inclusion of a mechanical connector that acts as a delay mechanism, which ensures that the new reinforcement does not take affect until after the wall has yielded; the delay mechanism can be as simple as a slotted connection. Figure 2 (b) shows a selective stiffness upgrade which is achieved by bonding steel plates to the wall across the plastic hinge region. The walls flexural strength is not increased by this intervention as the critical plane between the wall and foundation is not crossed by the steel plates. Figure 2 (c) shows a selective ductility upgrade which is achieved by applying U-shaped steel brackets on the wall edges with a through bolt to close the bracket. The ductility intervention works by increasing the level of confinement at the wall edges.



(a) (b) (c)

Figure 2: Basics of selective retrofit approaches:

a) Strength-only, b) stiffness-only, c) ductility-only (Elnashai 1992, Elnashai and Pinho, 1998)

3 THE CONCEPT OF SELECTIVE WEAKENING

Current seismic retrofit strategies generally focus on increasing capacity of individual elements or of the entire structure. A disadvantage of this existing approach is that it tends to lead to an increased demand on the structure as a result of the increased strength and stiffness. A selective weakening approach is herein being investigated/developed as it is believed that in some situations an initial strategic weakening of a structure will be a more appropriate option for achieving a successful seismic retrofit. Also by using selective weakening capacity design principles can be introduced to an existing structure that does not already exhibit them and a retrofit solution that results in minimal damage after a seismic response can be implemented. Acceptance of a "selective material removal" as a possible retrofit approach can be found in FEMA-273 (1997) and FEMA-356 (2000) documents. Preliminary suggestions in these documents included severing longitudinal reinforcement to change the response from a non-ductile mode to a more ductile mode or to segment walls to change their strength and stiffness.

The major advantages of using a complete selective weakening approach as proposed in this contribution for the seismic retrofit of structural walls include: a) reduce/control the demand on the foundation by controlling the capacity of the wall/s b) avoid the potential for buckling of longitudinal bars (by cutting them at the base) due to the large transverse reinforcement spacing in older building construction practice; c) introduce capacity design with the aim to improve the inelastic mechanism (e.g. from shear to flexure); d) reduce the damage connected with the development of a plastic hinge region, by enabling a controlled rocking motion to occur; e) further enhancing the response of the system by introduce a self-centring behaviour (i.e. no residual displacements) through vertical post-tensioning tendons as well as additional energy dissipation capacity through external mild steel or devices

Figure 3 shows the expected behaviour of an existing structural wall and different phases/options for a selective weakening retrofit. Figure 3 (a) shows the existing monolithic wall which is governed by a shear dominated inelastic mechanism as can be seen from the hysteretic response. Figure 3 (b) shows phase one of selective weakening which is termed "partial selective weakening" and two possible options for it application. The first option wall b' involves vertically splitting the wall, this changes the inelastic mechanism from shear to flexure but due to the large spacing of transverse reinforcement in the existing wall, bar buckling effects are expected in the hysteretic response. Material damage will also naturally develop in the plastic hinge region depending on the type of reinforcement and bond conditions (i.e. deformed or plain round bars, lap splices etc.) The second partial selective weakening option wall b'' involves a horizontal cut at foundation which will result in a rocking and re-centring behaviour.

Figure 3 (c) shows the second phase which is a full selective weakening, the term "full selective weakening" relates to a complete retrofit solution being developed that targets a specific level of strength/stiffness after an actual weakening intervention. A full selective weakening may therefore involve initially splitting the wall vertically and cutting it horizontally at foundation level, but then, in a second phase, introducing post-tensioning, energy dissipation devices or implementing other currently available retrofit techniques to re-enhance the properties of the structure to a target level. This may result in a retrofitted wall of equal or greater stiffness/strength/ductility than the original wall. Similarly, when protection from excessive seismic demand to other element (i.e. foundation) is a concern, the fully weakening intervention might target a level of strength lower than the original as-built solution. Wall (c') shows a wall that has been split vertically, cut horizontally at foundation level and un-bonded post-tensioning has also been introduced to control the rocking behaviour and increase the strength. The resulting hysteretic response is bilinear elastic which ensures a self-centring behaviour. Wall c'' has the basic properties of the wall c' with the addition of energy dissipation devices to increase both strength and dissipation capacity. As a result a "flag shaped" hysteresis, typical of recently developed high-performance seismic resisting systems based on ductile jointed (hybrid) connections (Priestley et al., 1999; fib 2004; Pampanin 2005a).

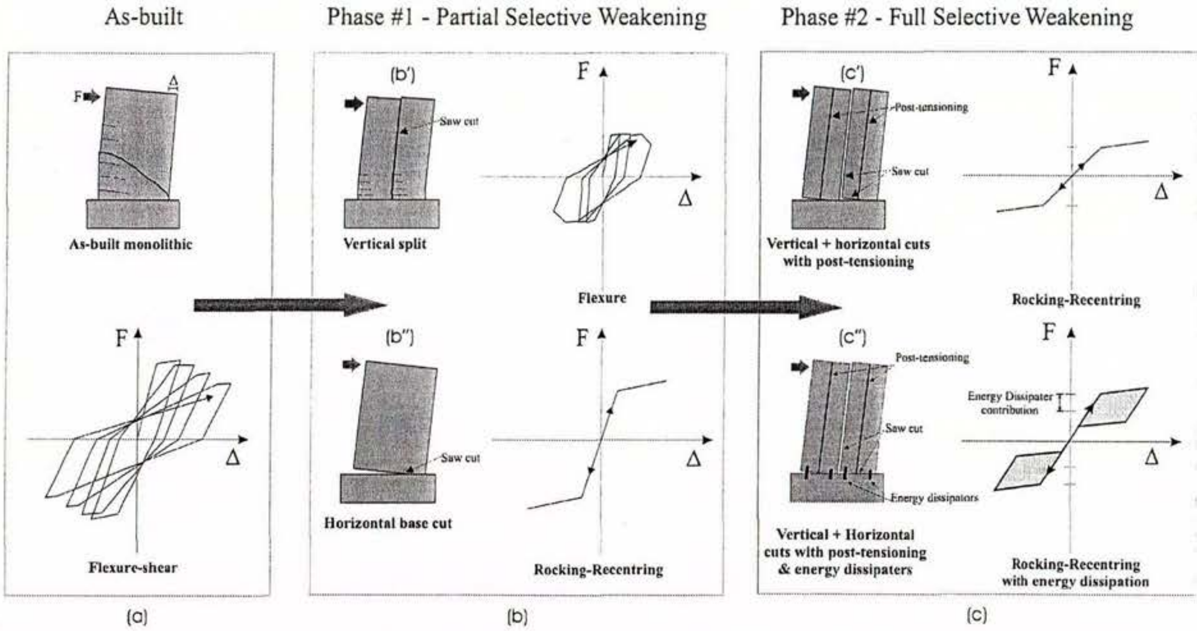


Figure 3: Expected damage and hysteresis structural response before and after intermediate phases of the retrofit intervention: (a) as-built wall; (b) partial selective weakening; (c) full selective weakening

3.1 **Modification to the demand-capacity balance within a selective weakening approach**

It is counter intuitive to think that by weakening a structure the seismic performance can be improved but this can result by a change in the inelastic mechanism or by a reduction in demand resulting from weakening. Selective weakening focuses on strategically altering the structural properties which will involve initially weakening the wall and then the possible option of upgrading the wall to meet a targeted performance limit. The targeted performance level may be weaker, similar or stronger than the original wall. An advantage resulting from weakening is that the demand on the wall is lowered as the strength/stiffness decreases. Demand on a structure as a result of seismic excitation is commonly expressed in terms of spectral acceleration which is usually found from design code acceleration spectra, by using a selective weakening approach that results in reduced strength/stiffness the natural period of vibration of the structure increases which in turn leads to a reduced demand.

When an as-built monolithic wall (Fig. 3a) is partially selectively weakened by a vertical split (Fig. 3 b'), this results in a stiffness of the two wall system that is about a quarter of the as-built wall and the natural period that is approximately double. A common property of acceleration spectra such as that found in NZS 1170.5 is that after 0.4 sec there is a steady reduction in the spectral acceleration. The resulting effect of increasing the natural period is that the demand significantly reduces as can be seen in Figure 4. The reduction in demand can also aided by a change in the inelastic mechanism which allows a higher level of ductility to be achieved, which in turn increases the level of damping and further lowers the spectral acceleration. A side effect of the reduced strength/stiffness and increased natural period that results from selective weakening is that the spectral displacement is increased. This can be seen in Figure 4 in terms of a displacement spectrum. A increase in damping due to a changing the inelastic mechanism can help to reduce the spectral displacement but also as a trade-off wall designs are commonly governed by minimum reinforcement requirements to resist temperature and shrinkage effects. This means that they will have a stiffness/strength in excess of that required so after selective weakening the displacements may still be within acceptable levels. Selective weakening will not always result in a overall weakening of the wall system, when "full selective weakening" is used a target performance level can be set to ensure that there is no demand increase or to control the level of demand increase.

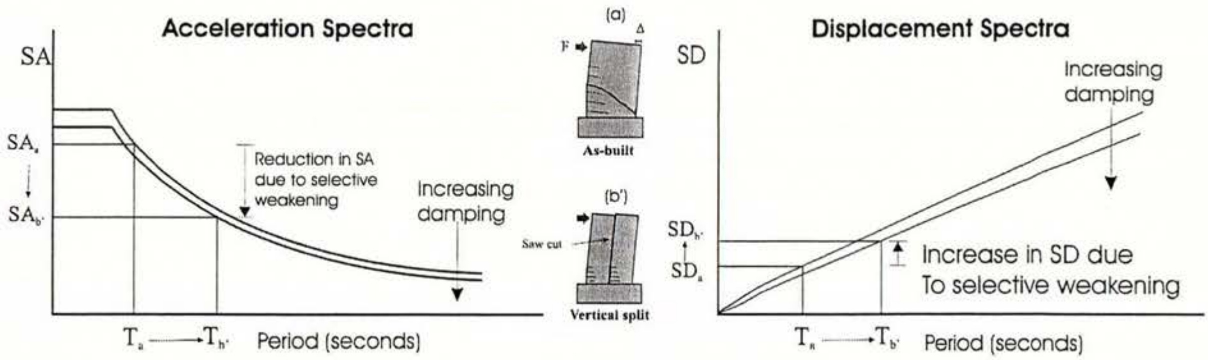


Figure 4: The effect of partial selective weakening on spectral acceleration and displacement demand

Figure 5 shows two possible examples of how selective weakening can be used for the capacity re-design of a wall and foundation system. Figure 5a shows force displacement responses for the as-built wall, a wall retrofitted by a conventional technique and a wall retrofitted by partial selective weakening. The as-built wall is governed by a shear dominated inelastic mechanism and a conventional retrofit technique formed using FRP wrapping is used as the conventional retrofit technique but this can increases the walls capacity so that the wall is stronger than the foundation. Wall b' is partially selectively weakened by a vertical split which results in a flexural inelastic mechanism developing in the wall and a wall with a capacity less than that of the foundation.

Figure 5b shows the force displacement response of the as-built wall which is governed by a shear dominated inelastic mechanism and two possible retrofit options using full selective weakening. Wall c' is selectively weakened by vertically splitting the wall, cutting it at foundation level and adding post-tensioning. This results in a rocking and re-centring behaviour with a lower strength than the as-built wall and the foundation. Wall c'' is the same as wall c' except that energy dissipaters have been added and the post-tensioning force has been increased. In this case a full selective weakening technique is used that targets a strength higher that the as-built wall but lower than the foundation capacity.

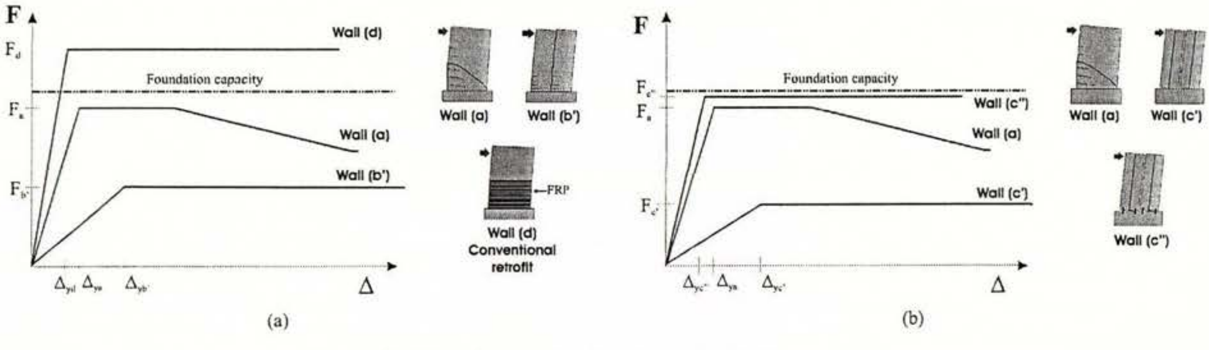


Figure 5: Selective weakening capacity design examples

4 IMPLEMENTATION OF SELECTIVE WEAKENING

As mentioned earlier the initial investigations into selective weakening will focus on implementing it for structural wall buildings, this will involve strategically saw cutting the walls vertically and/or horizontally. Horizontal saw cuts at foundation level will be used to sever longitudinal reinforcement to allow for rocking type behaviour while vertical saw cutting will be used to segment the wall into portions with a lower moment capacity and lower the chance of a shear dominated inelastic mechanism. But cutting the walls introduces a series of new issues that need to be considered and overcome to achieve a successful retrofit. Major problems introduced and possible simple solutions are: a) segmenting a wall by a vertical cut will involve cutting through the transverse reinforcement, a solution to reintroduce confinement and shear capacity such as FRP wrapping or steel confining plates will be needed; b) A horizontal cut at foundation level will sever the longitudinal reinforcement, therefore a solution to increase the moment capacity and energy dissipation will needed, this could include post-tensioning and damping devices; c) A horizontal cut could result in the wall sliding on the cut region therefore a mechanical shear key will be required; d) The interaction between the wall and floor diaphragm need to be considered.

5 EXPERIMENTAL INVESTIGATIONS

To validate selective weakening as a viable retrofit strategy a series of experimental investigations are being performed in the Civil engineering laboratory at the University of Canterbury. The experiments are being performed on 2/3 scale rectangular cantilever wall specimens with reinforcement details typical of those found in existing buildings. The experimental investigations also serve as a means of confirming analytical predictions, highlighting problems to overcome and are a chance to test which other currently available retrofit techniques are best suited for use in conjunction with selective weakening.

The first phase of experiments will be in relation to existing walls with plain round reinforcing bars and a straight lap detail. A control specimen will be tested and then a second wall with the same reinforcing details will be selectively weakened. As it is thought that the lap detail will govern the inelastic behaviour, selective weakening will be performed by horizontally cutting it at foundation level which will allow the wall to rock. Figure 6 below shows the reinforcement details of the first wall in the experimental program.

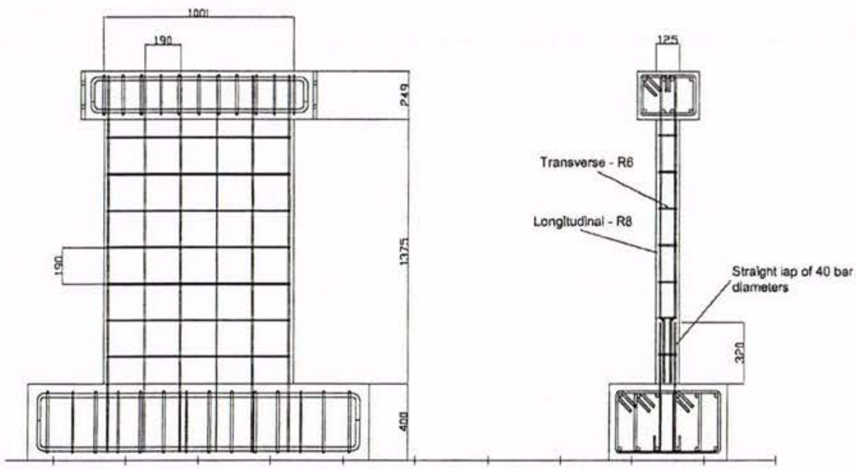


Figure 6: Experimental specimen with plain round reinforcing bars and a straight lap detail.

The second phase of experimental investigations is going to be performed on a wall with deformed reinforcement and a lap detail that does not govern the inelastic behaviour. A control specimen will be tested and the on a second specimen selective weakening will be performed by vertically cutting the wall into two segments.

6 CONCLUSIONS

In this contribution a preliminary feasibility study for the development of selective weakening as a seismic retrofit strategy for reinforced concrete structural walls has been outlined. Current retrofit strategies generally focus on increasing capacity but in certain situations a reduction in strength or weakening may be more appropriate. One of the major advantages is the ability to introduce capacity design to existing structures that do not already exhibit it. By selective weakening capacity design is not only limited to ensure that a flexure dominated inelastic mechanism is achieved before a shear dominated inelastic mechanism forms. Further, the design process can be comprehensive by ensuring that the foundation capacity is not exceeded. Selective weakening can also be used to implement recent technological developments in building systems. Such systems commonly utilise rocking behaviour to ensure minimal damage and a self-centring behaviour so that there are no residual displacements after seismic response.

Initial experimental investigations are on-going in the Civil Engineering laboratory at the University of Canterbury, which consist of 2/3 scale reinforced concrete walls with similar reinforcement details to those found in existing pre-1970s buildings in New Zealand.

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