NZSEE PCEE 2011

Proceedings of the Ninth Pacific Conference on Earthquake Engineering
Building an Earthquake-Resilient Society
14-16 April, 2011, Auckland, New Zealand

Displacement-based seismic retrofit design for non-ductile RC frame structures using local retrofit interventions at beam-column joints

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ABSTRACT: Seismic retrofit design using local retrofit interventions is often done using piece-meal iterative approach, in which the local retrofit design and global structural response are derived from iterative numerical models. Adopting a displacement-based seismic retrofit approach and following capacity-design principles, the effects of local retrofit interventions' can be correlated to the expected global deformation responses. Clearly acknowledging that displacement (or drift) is a better response parameter for structural and non-structural damage, a displacement-based methodology gives a more direct and rational seismic retrofit design. This paper will first introduce the concepts of displacement-based seismic retrofit. Then, the design procedure is illustrated for two local retrofit interventions for RC frames: a) selective beam-weakening retrofit, and b) post-tensioning retrofit and fibre-reinforced polymer jacketing. The design procedure is then verified using non-linear time-history analysis on a case study building retrofitted using the two local interventions.

1 INTRODUCTION

1.1 Seismic retrofit design – current practice

Seismic retrofit using local retrofit interventions is often designed using a piece-meal iterative approach, in which the local retrofit design and global structural response are derived from iterative numerical models. The lack of understanding of the direct correlation between the global structural performance enhancements and the associated local retrofit interventions encourages the widespread use of global strengthening techniques (e.g. new shear walls, new braced-frames or seismic isolation) (Thornton 2010).

The state-of-the-art guidelines on the seismic retrofit design outline several different approaches for the seismic retrofit design of non-ductile reinforced concrete (RC) frames. The NZSEE guidelines (2006) gives detailed force-based and displacement-based assessment procedures for RC buildings. The force-based approach is developed from Park (1996) static force-based capacity approach while the displacement-based approach is adopted from Priestley (1995). While both approaches focus on achieving capacity design and the desirable ductile failure mode, the design approaches are not correlated to any damage parameters (e.g. inter-storey drift, θ_d). The NZSEE guidelines also do not specify any guidance for a performance-based retrofit outcome but the guidelines provide values of maximum allowable strains for various materials.

The American / ASCE approach is based on a performance-based seismic assessment using numerical modelling (either elastic or non-linear and either static or dynamic analyses), consistent with the practitioners' approach for new building design (ASCE-SEI-41-06 2007). The performance of the retrofitted structure, however, is assessed post-analysis and piece-meal iterative approach is necessary to achieve an optimal retrofit design. For example, Chambers *et al.* (2007) describes an integrated approach in which the ASCE-41 deformation acceptance criteria (e.g. plastic rotation for columns) are incorporated within a non-linear dynamic analysis.

Adopting a displacement-based seismic retrofit approach and following the capacity-design principles, local retrofit interventions' effects can be correlated to the expected global deformation responses.

Acknowledging that displacement (or drift) is a better response parameter for structural and non-structural damage (ASCE-SEI-41-06 2007), a displacement-based methodology gives a more direct and rational seismic retrofit design (Priestley *et al.* 2007). This paper outlines a simplified displacement-based seismic retrofit design procedure for regular non-ductile RC frames without masonry infill walls for the conceptual/preliminary design. The design principles can be subsequently extended for more complex structural forms. The design procedure is implemented on a case study pre-1970s RC frames building and verified using non-linear time-history analysis.

1.2 **Performance-based seismic retrofit response parameters**

Performance-based seismic retrofit allows a greater flexibility in deciding building performance objectives for seismic retrofit when compared to the design of new buildings. The required global performance objectives can be correlated with the building structural and non-structural performance levels (PLs) at the given levels of seismic intensities associated with specified return periods. This is illustrated in Table 1, using the ASCE-41 terminology for the three global performance objectives: (a) to maintain functionality of the building post-earthquake (Enhanced Rehabilitation Objective - ERO), (b) to minimise fatalities (Basic Safety Objective (BSO)) or (c) to prevent collapse (Limited Rehabilitation Objective - LRO).

Table 1. Seismic retrofit performance-objective matrix and performance levels based on the ASCE-41.

ASCE-41 (2006) Table C1-1. Rehabilitation Objectives (adapted)		Target Building Performance Levels			
		Operational PL	Immediate Occupancy PL	Life Safety PL	Collapse Prevention PL
Earthquake Hazard Level	50% in 50 years	a	b	c	d
	20% in 50 years	e	f (100%NBS)	g (67%NBS)	h (33%NBS)
	10% in 50 years	i	j (100%NBS)	k (67-100%NBS)	l (33%NBS)
	2% in 50 years (MCE)	m	n	o	p (67-100%NBS)

Notes: Limited Retrofit Objectives (LRO)
Basic Safety Objectives (BSO)
Enhanced Retrofit Objectives (ERO)
%NBS - percentage of new building standard (NZSEE,2006)

PL = Performance Level
MCE = maximum credible earthquake

The different performance levels can be associated with displacement (or acceleration) demand parameters, as well as non-failure of critical structural elements. Typical code provisions (e.g. ASCE-41 and NZS1170 (2004)), specify inter-storey θ_d limit of 2.0-2.5% for the design earthquakes as the BSO (or 100% New Building Standard (NBS) in NZSEE terminology). The definition of the deformation limit states can be also based on local structural elements deformation capacities (e.g. moderate joint cracking at θ_d = 1.0%) and/or non-structural elements deformation capacity (e.g. moderate cracking of clay-brick infill walls at θ_d = 0.5%) (ATC-58 2009).

In the 2004 New Zealand Building Act (DBH 2004) and the NZSEE guidelines, the adoption of the quantifiable minimum standard of 33% of NBS is an attempt towards achieving LRO for critical earthquake-prone buildings. It represents a reasonable balance of imposing a requirement for all non-complying buildings (<100%NBS) and the existing status (in which only unreinforced masonry buildings are retrofitted). However, achieving the minimum standard of 33% NBS does not imply collapse prevention PL. The %NBS parameter defined in terms of lateral strength is yet to be correlated to quantifiable performance (e.g inter-storey drift, damage etc). As such, it is of paramount importance that in addition to achieving the minimum %33 NBS lateral capacity, ductile failure mechanism (with sufficient ductility capacity) must also be attained for collapse prevention.

Alternatively, as shown in Table 1, different PLs (based on quantifiable damage parameter e.g. θ_d) can be specified for LRO, depending on the risk averseness of the owners and community. The θ_d limit of 3.0-4.0% is typical given as the collapse prevention deformation limit state.

2 DISPLACEMENT-BASED SEISMIC RETROFIT DESIGN

2.1 Previous works

A direct displacement-based seismic design and assessment framework (abbreviated as DDBD) has been previously introduced by Priestley and colleagues (1997; 2007). This approach was modified and adopted within the NZSEE guidelines (2006). Within the NZSEE guidelines, the retrofit design is attained by comparing the displacement capacity derived from non-linear pushover analysis against the required displacement demand derived from a desired %NBS, the assumed ductility (μ) and the associated equivalent viscous damping (ξ_{sys}).

Marriott *et al.* (2007) recently extended the DDBD approach for global strengthening of pre-1970s RC frames with rocking walls retrofit solution. The added rocking walls retrofit limits the displacement demand of the RC frames and prevents brittle failure modes. The target displacement limit is defined at the start of the design procedure to correspond to the required PLs.

2.2 The proposed displacement-based seismic retrofit design

In principle, the proposed displacement-based retrofit design aims to determine the appropriate level of retrofitted members' capacities in the beam-column joints for a selected seismic retrofit performance objective. The basic retrofit design strategy is to control the hierarchy of strength of the beam-column joint to respect capacity design principles and to attain ductile flexural beam-hinging failure mode.

Given the hierarchy of strength of the as-built beam-column connections in the RC frames, depending on its typology, geometry, reinforcing details etc., various local retrofit techniques can be applied to modify the hierarchy of strength within the M-N (moment-axial force) performance domain of the connection to achieve the targeted performance. The proposed four-step procedure is illustrated in Figure 1. The following sub-sections will highlight some of the key assumptions while further details of the procedure are described in the Chapter 3 of ref. (Kam 2010).

3 STEP-BY-STEP RETROFIT DESIGN PROCEDURE

3.1 Step 0: Define the target performance objectives

The targeted performance objectives for the retrofitted building can be defined in relative to the clients' and building codes' requirements, in terms of engineering demand parameters (such as θ_d). It is important to recognise that the elimination of non-ductile failure mechanisms and critical structural weaknesses (e.g. irregularity) are more significant than the upgrade of the lateral force capacity alone. In New Zealand practice (e.g. the NZSEE guidelines), the definition of seismic retrofit objective based on the desired building base shear as a function of %NBS does not correlate well with expected damages or global seismic response. As such, the %NBS parameter can be re-defined in terms of θ_d and the associated expected performance. For instance, the 33%NBS limit can be defined as LRO, thus collapse prevention PL (θ_d = 3.5-4.0%) in the design earthquakes (although this assumption is principally incorrect as discussed in Section 1.2).

3.2 Step 1: Defining retrofitted-frames DDBD parameters

The DDBD parameters would define an equivalent single-degree-of-freedom (SDOF) elastic system to the retrofitted RC frames, with the secant stiffness, K_{eff}, to the target displacement, Δ_u, at the effective height, h_{eff}, (Figure 1-step 1). The proposed retrofit design overcomes the two important difficulties of the direct displacement-based seismic assessment: a) which element of the structure will first fail or govern and b) what is the corresponding displacement profile of the building (Priestley *et al.* 2007).

By the virtue of the capacity design and the hierarchy of strength check of the beam-column joints, a beam-sway mechanism can be achieved by design. As such the deformed shape, yield displacement, Δ_y and damping-ductility (ξ_{sys}-μ) formulations for a flexural hinging beam-sway RC frame can be adopted with minor modifications. Other parameters such as the effective mass, m_{eff}, and h_{eff} would be a function of the building properties. The following expressions are used to generate the DDBD parameters for a beam-hinging RC frame, but further detail is available in ref. (Priestley *et al.* 2007):

$$\Delta_i = \frac{\Delta_u}{H_c}\left(\frac{\delta_i}{\delta_1}\right) \text{ where } \delta_i = \begin{cases} \dfrac{H_i}{H_n} & for \quad n \le 4 \\ \dfrac{4}{3}\dfrac{H_i}{H_n}\cdot(1-\dfrac{H_i}{4H_n}) & \quad\quad n > 4 \end{cases} \tag{1}$$

$$\Delta_{u,\vartheta_c} = \frac{\sum m_i \Delta_i^2}{\sum m_i \Delta_i} \qquad m_{eff} = \frac{\sum m_i \Delta_i}{\Delta_{d,u}} \qquad h_{eff} = \frac{\sum m_i \Delta_i h_i}{\sum m_i \Delta_i} \qquad \text{(2), (3) and (4)}$$

where H_c is the inter-storey height at the base of the building; H_i and H_n are the height of level i and roof height; Δ_i, δ_i, and m_i, are the displacement, shape factor and mass at level i.

The ξ_{sys} for the SW-retrofitted pre-1970s frames is hard to be determined at the preliminary stage. Tentatively, a low, constant with ductility, value of ξ_{sys} = 12.5% can be assumed based on the experimental results (Kam *et al.* 2010). The use of constant ξ_{sys} removes the need to estimate Δ_y and μ. Δ_y may be difficult to be estimated for the SW-retrofitted pre-1970s RC frames with plain round bars because the elastic deformations of the beams, columns and joints are hugely affected by the bond capacity of the reinforcement. The use of constant ξ_{sys} may be non-conservative however for limited-ductility design. Alternatively, the expressions for unbonded post-tensioned precast concrete systems (ξ_{sys} ranges from 5-18%) can be used for SW retrofit solutions with external post-tensioning (Marriott *et al.* 2007; Priestley *et al.* 2007).

3.3 Step 2: Determine the effective period and the required base shear.

The displacement response spectrum is used to derive the required effective period, T_{eff}, corresponding to the target design displacement, $\Delta_{u,\theta d}$, given the level of damping (ξ_{sys}). This is illustrated in Figure 1-step 2, where the NZS1170:5 (2004) displacement hazard spectra is used in conjunction with three performance levels: a) Limited Performance, LP (θ_d =3.0%), b) Basic Performance, BP (θ_d =2.0%) and c) Advanced Performance, AP (θ_d =1.0%). The 5%-damped elastic hazard spectra ($S_{d,elastic}$) are reduced using a damping reduction factor, η:

$$S_d\left(\eta\{\xi_{sys}\}\right) = S_{d,elastic}\cdot\eta \qquad \text{where} \qquad \eta = \left(\frac{0.07}{0.02+\xi_{sys}}\right)^{\alpha} \ge 0.7 \qquad \text{(5) and (6)}$$

where α = 0.25 and 0.50 for near field and far field design ground motions respectively. Thus, the required based shear of the SW-retrofitted frames to achieve the previously defined target performance-objective (in θ_d) is calculated as $V_{b,req} = K_{eff} \cdot \Delta_{u,\theta d}$, where K_{eff} is given by:

$$K_{eff} = 4\pi^2 \frac{m_{eff}}{T_{eff}} \Delta_{u,\theta_d} \tag{7}$$

3.4 Step 3: Distribute the base shear and determine required members strength.

Given the V_{b-req}, the required flexural strength of beam hinges, $M_{b,req}$, can be determined using an equilibrium approach (Priestley *et al.* 2007) or structural analysis with the V_{b-req} distributed up the building height. The base shear is distributed in proportion to the floor mass and displacement, with an additional 10% applied to the roof level to account for higher mode effects:

$$F_{roof} = 0.1V_b + 0.9V_b \frac{m_{roof}\Delta_{roof}}{\sum m_i \Delta_i} \text{ at roof level} \qquad F_i = 0.9V_b \frac{m_i \Delta_i}{\sum m_i \Delta_i} \text{ at other level } i \qquad \text{(8) and (9)}$$

A close form equilibrium distribution described by Priestley *et al.* (Priestley *et al.* 2007) is used:

$$V_{b,i} = N_e \frac{V_{storey,i}}{\Sigma V_{storey,i}} \quad \text{and} \quad N_e = \frac{\sum (F_i h_i) - V_b (0.6 H_c)}{L_{building}} \tag{10 and 11}$$

where N_e is the earthquake induced tension force in the ground columns (or sum of beam shears) and $V_{storey,i}$ is the storey shear at level i. $V_{storey,i}$ is the cumulative applied distributed base shear force (F_i). $L_{building}$ is the length of the building (sum of all bay lengths, L_b). The required beam flexural capacity at each level i for the given θ_d is then given by $M_{b,i,\theta}=V_{b,i} / L_b$.

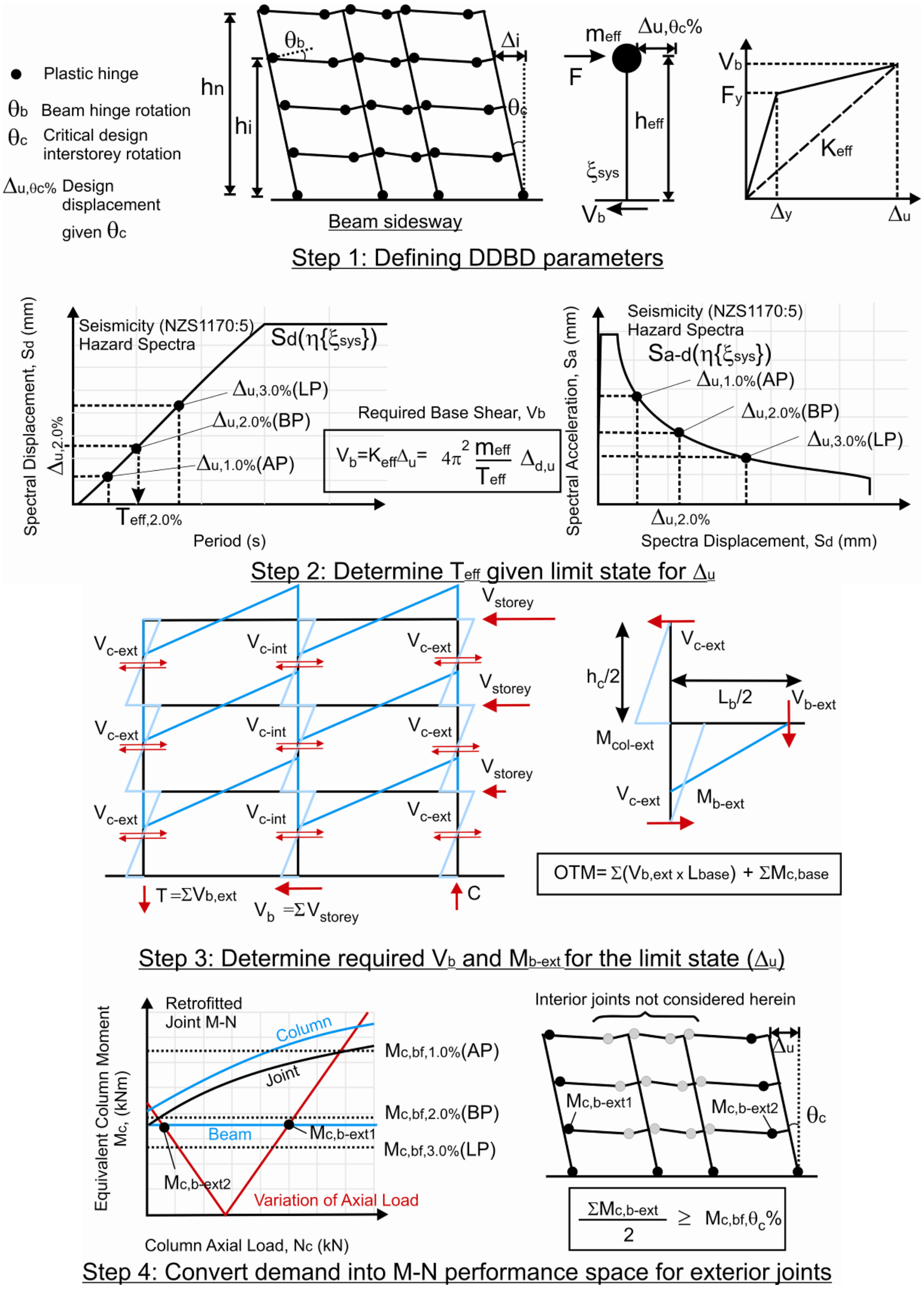

Figure 1. The proposed displacement-based seismic retrofit design for non-ductile RC frames.

The internal force distribution depicted in Figure 1-step 3 assumes the interior joints have sufficient strength to develop $V_{c\text{-}int}$ in the interior columns. If the interior joints have insufficient strength/ductility/deformation capacities, then the interior joints will need to be retrofitted as well. In the scenario where high θ_d level (e.g. LRO or BSO) and a mixed beam/column-sway inelastic mechanism are acceptable, the interior joints and columns are only checked for its ductility and deformation capacities. Alternatively, the designer may opt to allocate higher column shear demands to the exterior columns, in order to achieve a θ_d performance level.

3.5 **Step 4: Conversion of demand into a M-N performance domain.**

Lastly, for the given $M_{b,i,\theta}$ at the exterior beam spans to sustain the beam-sway mechanism at a given seismicity, $M_{b,i,\theta}$ can be converted into an equivalent column moment, $M_{c,bf,\theta}$. Therefore, the $M_{c,bf,\theta}$ for various performance levels can be projected and compared within the M-N performance space of the retrofitted exterior beam-column joints (Figure 1-step 4). For the local retrofit design within the M-N domain, in addition to the need to satisfy the hierarchy of strength requirement, it is also necessary to satisfy the required flexural strengths of exterior spans' beams, $M_{c,b\text{-}ext}$, to achieve the required performance level ($\sim M_{c,bf,\theta}$):

$$\frac{\sum M_{c,b-ext}}{2} \geq M_{c,bf,\theta} \quad \text{and} \quad \frac{\sum M_{c,bf}}{2} \geq \frac{\sum M_{c,b-ext}}{2} \qquad \text{(12) and (13)}$$

where $M_{c.bf}$ is the provided beam flexural capacity (in terms of M_c). As explained in §2.4.1, the following expression can be used to convert strength value of various failure modes into the equivalent column moments (M_c). $M_{c,bs}$ and $M_{c,j}$ are the provided beam shear capacity and provided joint shear capacity in terms of M_c.

$$M_{c,bf} = M_b \frac{L_b}{H_c}\frac{H'_c}{L'_b} \quad M_{c,bs} = V_b \frac{L_b}{H_c} H'_c \quad M_{c,j} = \frac{V_{jh} H'_c \, jd}{(H_c - h_b - jd)} \qquad \text{(14), (15) and (16)}$$

where H_c, H'_c, L_b and L'_b are geometry parameters as illustrated in Figure 2.

Equations 12 and 13 assume the moment demands in the flexural hinges at the exterior spans have sufficient ductility to allow moment redistribution during an earthquake. As one exterior span goes into positive moment, the other will go into negative moment. Therefore, it is not unreasonable to consider the sum of the exterior spans' beam flexural capacity in computing the total contribution from the exterior beam-column joints.

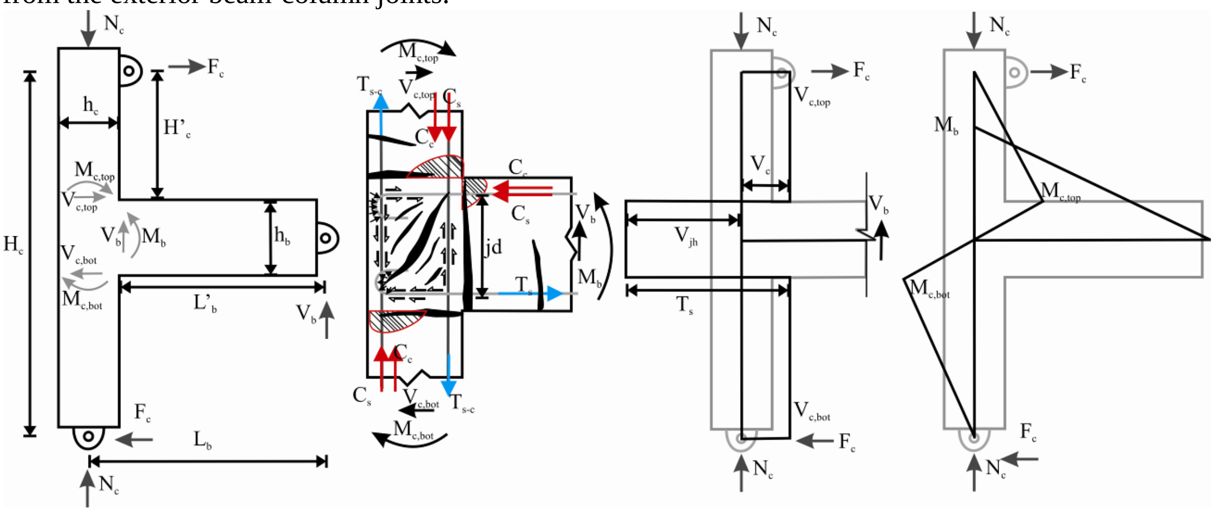

Figure 2. Equivalent column moment for various internal actions on exterior beam-column joints subassembly: a) Labelling; b) Internal action of exterior joint; c-d) Corresponding shear force and bending moment diagram of exterior beam-column joint subassembly.

4 SEISMIC RETROFIT DESIGN EXAMPLES

4.1 Case study building

A case-study six-storey three-bay RC frames building is designed to simulate the typical pre-1970s mid-rise residential/commercial multi-storey building. Poor material properties, deficiencies in reinforcement detailing and violation of capacity design philosophy are intently included in the prototype structure. The global geometry and the 2nd floor beam-column joint geometry are shown in Figure 3a and further information is available in ref. (Kam 2010).

The columns are tapered from 15” (380mm) squares at first two floors to 14” (350mm) squares at upper stories. The beams are 19.5” (495mm) deep by 13.75” (350mm) wide. Column stirrups are typically 3/8” bars at 6” spacing (i.e. 9.5mm diameter at 150mm centres) while beam stirrups are 3/8” bars at 8” spacing (9.5mm diameter at 200mm centres). Beam-column joints are not reinforced with stirrups. The beam longitudinal bars are anchored into the exterior joints are using double 180° hooks for both the top and bottom beam reinforcements. The reinforcing details for the exterior beam-column joints are shown in Figure 3a (insert).

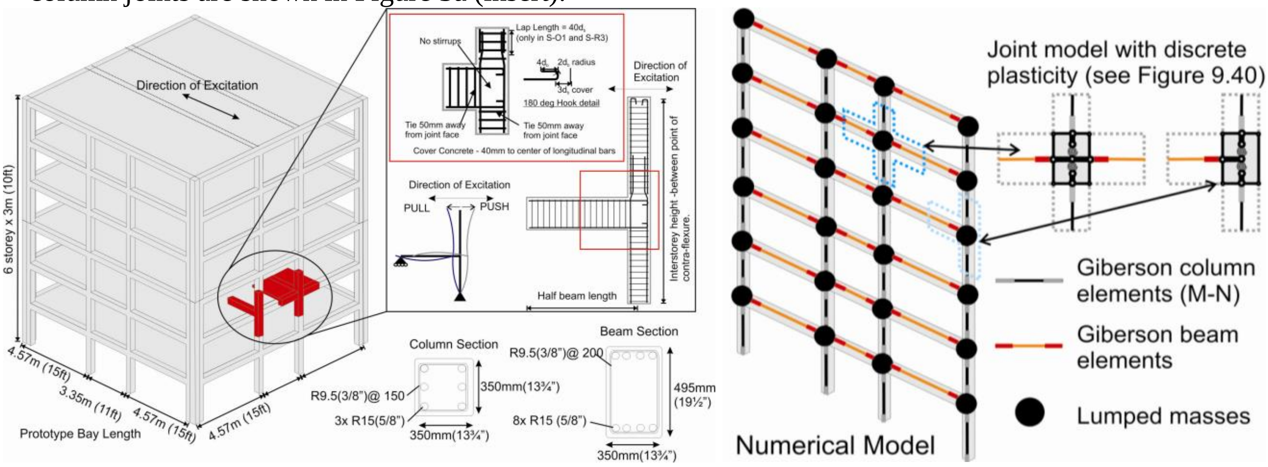

Figure 3. a) Geometry and structural detail of the case study RC frames building; b) Structural model of the prototype frame.

4.2 Numerical example of the seismic retrofit design

Table 2 summarises the DDBD parameters for the retrofitted six-storey case study building for a 2.0% design θ_d, based on Wellington seismicity (NZS1170 2004) with a peak ground acceleration (PGA) of 0.4g, S_p factor of 1.0 and an assumed system equivalent viscous damping ξ_{sys} of 10%. Soil class C and no near-fault amplification are assumed. This is but one retrofit design example and depends on different local retrofit interventions at the beam-column joints, different performance levels can be attained.

Table 2. DDBD parameters for the retrofit design of the six-storey prototype RC frame for the Wellington seismicity, with Z=0.4g, S_p=1.0, soil class C, N=1..` and an assumed system viscous damping, ξ_{sys}=10%.

SDOF design displacement		0.201	m	SDOF design interstorey drift		2.0	%
SDOF effective height		11.820	m	SDOF equivalent viscous damping		10.00	
SDOF effective mass		494	tonnes	SDOF design base shear (all frames)		934.6	kN
SDOF effective period		2.05	sec	SDOF base overturning moment		2906.3	kNm
Beam Design Moment	Level 1-2	93.6	kNm				
	Level 3-6	46.4	kNm	Column Design Moment	Interior L1-3	140	kNm
		Exterior Column 1	Interior Column		Exterior L1-3	70	kNm
Column Axial Load	Base: N_{G+Q+E}	90.0	1620.1		Interior L4-6	119	kNm
Column Axial Load	L3: N_{G+Q+E}	348.3	946.1		Exterior L4-6	59	kNm

The S_p factor is defined in the NZS1170:5 to account for additional conservatism in the design of ductile structure (when compared to brittle structure). The S_p factor is a function of the structural ductility, μ, ranging from 1.0 for $\mu = 1$ (elastic structure) to 0.7 for $\mu \geq 2$. For displacement-based retrofit design, a S_p of 1.0 is recommended and a more realistic ξ_{sys} can be used. However, in order not to unnecessary penalise the displacement-based retrofit design, the S_p as a function of μ can be used with a conservative estimation of the damping ξ_{sys}.

The case study building is retrofitted using two local retrofit interventions: (a) selective beam-weakening retrofit (R1) and (b) post-tensioning retrofit and fibre-reinforced polymer jacketing (R2). The local retrofit design at the exterior beam-column joints level must first satisfy the capacity design requirements of weak beams, strong columns and joints. This is achieved by satisfying the correct hierarchy of strength within each beam-column joint. The details of the design of the local retrofit interventions are beyond the scope of this paper. Design expressions for the selective-weakening and post-tensioning retrofit of pre-1970s beam-column joints (Kam 2010) and FRP jacketing retrofit (CEN 2006; Akguzel and Pampanin 2009) are available in literature. Figure 4a shows the details of the local retrofit interventions used in the case study building.

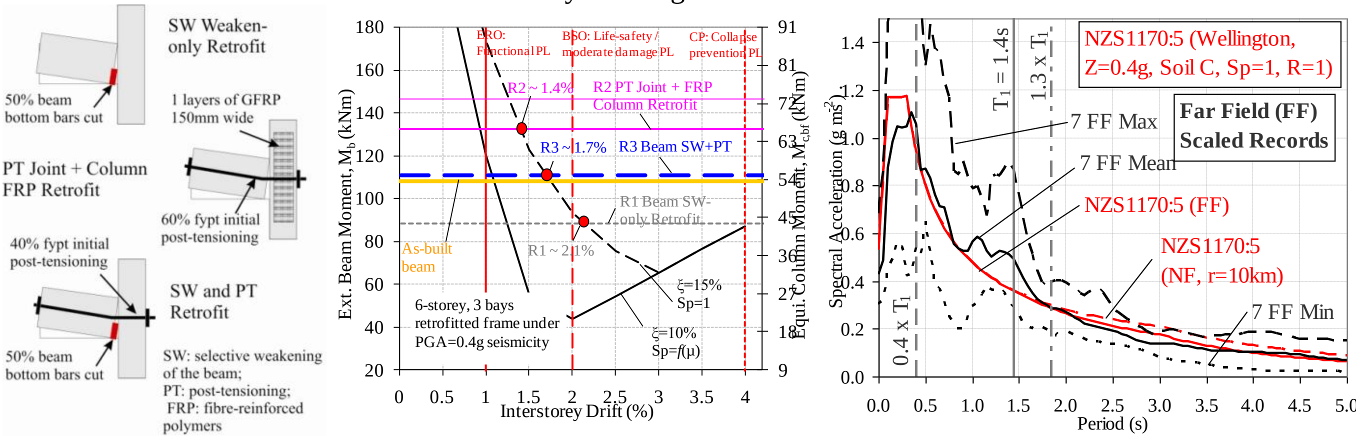

Figure 4. a) Schematic description of the three local retrofit solutions: R1, R2 and R3 designs; b) DDBD retrofit design for the prototype 6-storey 3-bay RC frames building under Wellington (PGA=0.4g) seismicity; and c) Spectral mean and maximum/minimum envelope for the scaled far-field records compared to the NZS1170:5 (2002) 5% damped design spectrum.

Figure 4b presents a series of design curves for several retrofit options, which relate the targeted design θ_d with the exterior beam-column joint's $M_{c.bf}$. For example, the R1 retrofit solution, which involves 50% beam bottom bars weakening, has a beam moment capacity, M_b of 88.5kNm (or 40.1kNm in terms of $M_{c,bf}$). This corresponds to a design θ_d of 2.1% for the prototype building, under the Wellington seismicity, a S_p factor of 1.0 and an assumed system equivalent viscous damping ξ_{sys} of 10%. Thus, Figure 4b illustrates how different local retrofit techniques, (beam-weakening, post-tensioning, FRP jacketing etc) can achieve different retrofit performance objectives and limit states. Figure 4b also highlights the large uncertainty introduced by the so-called Sp factor that is based on structural ductility.

5 NUMERICAL VALIDATION OF THE DESIGN PROCEDURE

5.1 Numerical models

Non-linear time-history (NLTH) analyses are performed using the finite-element program RUAUMOKO (Carr 2008). A Newmark-beta integration scheme with a 5% Rayleigh damping model proportional to the initial stiffness is adopted. P-delta effects are ignored. Lumped mass and lumped plasticity modelling are adopted, where inelastic deformations are limited to discrete inelastic rotational springs in the joints, beams and columns. The numerical model of the prototype frame is illustrated in Figure 3b. Further information of the numerical model is given in the Chapter 9 of ref. (Kam 2010).

Careful attention is given to the correct representation of the beam-column joint connection, using lumped plasticity rotational macro modelling approach (Pampanin *et al.* 2003). The Wayne-Stewart hysteresis rule is used to model cyclic strength degradation, stiffness degradation and pinching hysteresis behaviour, for the joint springs of the existing RC frames, as per Liu (2001) calibration.

As-built and retrofitted beam and column elements are modelled using the Giberson frame elements with thin modified Takeda hysteresis (α=0.5, β=0). In the SW-retrofitted frames' models, the inelastic properties of the beam, column and joint elements are changed to account for the local retrofit interventions. For the R1-retrofitted frame, the beam-weakening is modelled by a reduction of the beam negative moment capacity and the beam initial stiffness. For the R2-retrofitted frame, the un-bonded post-tensioning increases the beam and joint moment capacities, as well as the post-yield stiffness of the beam. These effects of the local retrofit interventions are based on laboratory test result observation (Kam *et al.* 2010).

5.2 **Ground Motions**

Seven scaled historical 'far-field' (without any directivity effect) strong ground motion records are used in the analyses. Similar analyses with a suite of near-fault earthquakes were also carried out in ref. (Kam 2010). The scaling of the earthquake records are done in accordance to the recommendation of the NZS1170:5 (NZS1170 2004). The prototype building's design site is assumed to be Wellington, with the peak ground acceleration of 0.4g and a probability of exceedance of 10% in 50 years (R=1.0). Soil class C is assumed. The response spectra of the scaled records are presented in Figure 4c.

6 **RESULTS**

6.1 **Non-linear time-history responses**

The non-linear time-history (NLTH) results are summarized in Table 3 and the average θ_d responses are presented in Figure 5. Figure 6 presents the mean of the peak response values of the global frame and the mean peak rotations of each modelled component (i.e. joints, beams and columns) under the seven far-field ground motions.

The as-built RC frame with slender columns and heavy infill partitions has a relatively long period (T_1~1.64s). The weakening of the beam reduced the beams stiffness and therefore softened and lengthened the period of R1-retrofitted building (T_1~1.73s), as one would expect from the beam-weakening-only retrofit. The post-tensioning of the beam-column joint in R2-retrofitted building, on the other hand, stiffened the overall structure (T_1~1.57s).

The as-built frame performed poorly with the average maximum θ_d of 2.43% at the 2nd level, with the joint shear deformation as the dominant inelastic mechanism, with moderate column hinging at the base and almost negligible beam plastic deformation. The average joint plastic rotation demand, exceeding 2.34% radians, suggested extensive joints damage, concrete spalling, column bars buckling and incipient structural collapse. The non-ductile base column also had significant rotational demand up to 1.18% radians.

The θ_d at the effective height of the building (2.0%) was comparable to the design θ_d of 2.1% for the R2-retrofitted building. While no significant improvement was observed in terms of the global displacement and θ_d responses for the R1-retrofitted frame, the beam-weakening retrofit effectively changed the dominant inelastic mechanism from a brittle joint shear failure to a relatively more ductile beam flexural hinge, as evident from Figure 6. As the beams rotational demands were less than 1.75% (in the maximum cases), ductile flexural response would have been achieved, as demonstrated in the laboratory tests (Kam *et al.* 2010). The inelastic demands in the base columns and the roof knee joints indicated moderate but repairable damage of these elements. Further retrofit interventions of these elements may be necessary.

The R2-retrofitted frame showed the improvement in the θ_d responses as well as in the components' inelastic deformation demands. While the inelastic demands within the beam, joint and column were more significant in the R2-retrofitted frame (see Figure 6c), the additional damping from these hinges reduced the global θ_d response marginally. The θ_d at the effective height of the building (1.54%) was comparable to the design θ_d of 1.7%.

Table 3. Summary of non-linear time-history analyses results.

Buildings	Design	Max inter-storey drift (%)	Effective height drift (%) *	Roof Drift (%) **
As-built frame	-	2.43%	2.08%	1.70%
R1-retrofitted frame	~2.1%	2.42%	2.00%	1.52%
R2-retrofitted frame	~1.7%	1.97%	1.54%	1.14%

Note: * Effective height of the building is 11.82m, as calculated based on the DDBD expressions.
** Roof drift = roof displacement / building height.

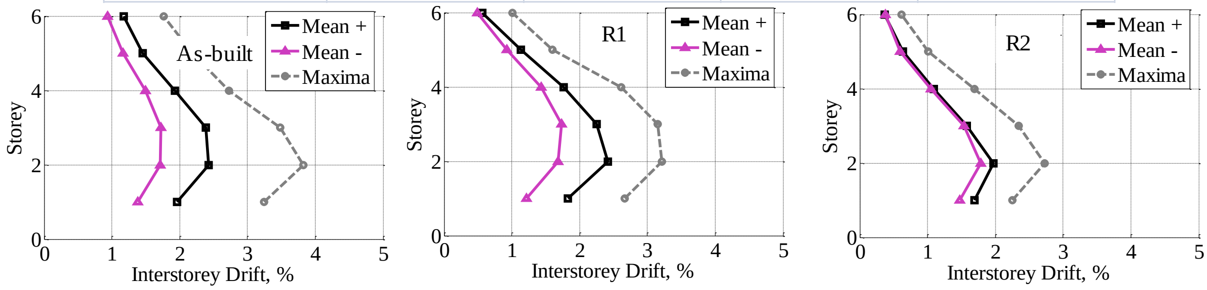

Figure 5. Average of peak inter-storey drift, θ_d, responses: Column: a) As-built frame; b) Beam-weakening only R1 retrofitted-frame; and c) Full SW R3-retrofitted frame.

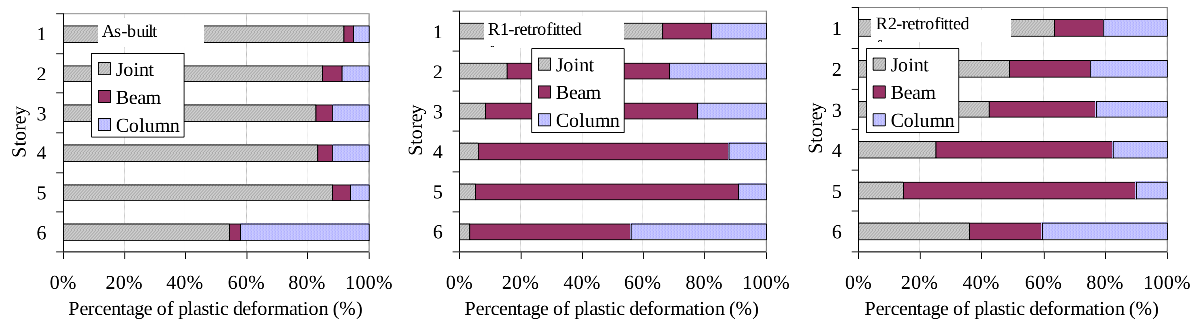

Figure 6. Decomposition of the plastic deformations for the three models under far-field ground motions: a) As-built; b) Beam-weakening only retrofit (R1); and c) External post-tensioning retrofit (R2).

The θ_d responses were generally higher than the average drift at the effective height, as the NLTH responses' deformed shape was not the beam-sway deformed shape assumed in the displacement-based retrofit design procedure. As illustrated by the distribution of θ_d up the building height in Figure 5, higher deformation demands were observed in the lower two to three storeys. One critical reason for this was the simplified design adopted for this study, in which the retrofitted beams and the columns capacities were not varied up the building height. The design assumed the ductility demands on the plastic hinges could be redistributed in the retrofitted frames during the earthquakes.

6.2 Limitations

While the aim of attaining ductile beam flexural failure mode, as per the capacity design philosophy was successful, evident from the results shown in Figure 6, the global inter-storey θ_d responses exceeded the design expectations in some cases.

The deformed shape of the building depends heavily on the assumptions of the distribution of the inelastic mechanisms. The use of a constant beam flexural strength up the building elevation and the reliance on possible moment redistribution may not be suitable for the seismic retrofit design of non-ductile RC frames. As shown in Figure 5, this design configuration leads to significantly higher inelastic demand at the lower storeys and limited inelasticity at the upper storeys. This results in a less-than-expected flexural inelastic action and damping, and therefore, higher global responses in terms of displacements and drifts.

Further studies in correlating the retrofit design to the deformed shape of the retrofitted frame buildings are necessary to refine the design procedure.

7 CONCLUSIONS

A displacement-based design procedure to derive the lower bound of required retrofitted elements capacities given a targeted performance level is presented. Two local beam-column joint retrofit interventions, namely a) selective beam-weakening retrofit, and b) post-tensioning retrofit and fibre-reinforced polymer jacketing, are used to demonstrate design procedure for a case study pre-1970s RC frames building.

This conceptually straight-forward retrofit design approach can be implemented in a spreadsheet program for preliminary retrofit design. The advantage is a direct correlation with seismic performance response parameters such as the θ_d and related structural/non-structural damages to the retrofitting design decisions. Incorporating the local seismic retrofit interventions design from a global level at the conceptual stage allows for a more efficient if not economical retrofit solution.

However, the NLTH results have shown the limitations of the some of the assumptions including the deformed shape profile and the moment redistribution of the retrofitted frames. Further parametric analyses on different building configurations and scenarios are required to improve the robustness of the simple design approach.

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