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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.

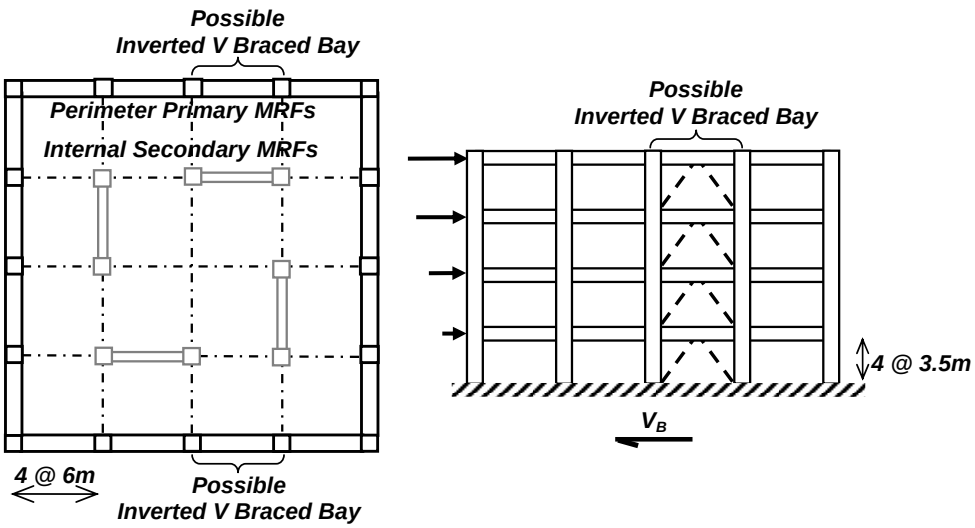


Figure 1: Plan and elevation of study building showing primary moment-resisting frames (MRF) and internal secondary MRF positions. Also shown are the bays used for the buckling restrained brace frames (BRBF) with inverted V form.

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

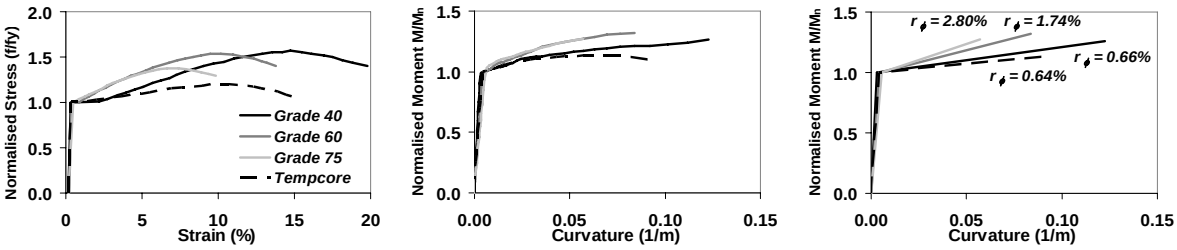
Previous studies [MacRae and Kawashima, 1997; Kawashima et al., 1998; Borzi et al., 2001; Pampanin et al., 2002] have shown that the post-yield stiffness to initial stiffness ratio is the principal factor governing the residual deformation response of a structure. Physically P-Δ is the primary influence 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)

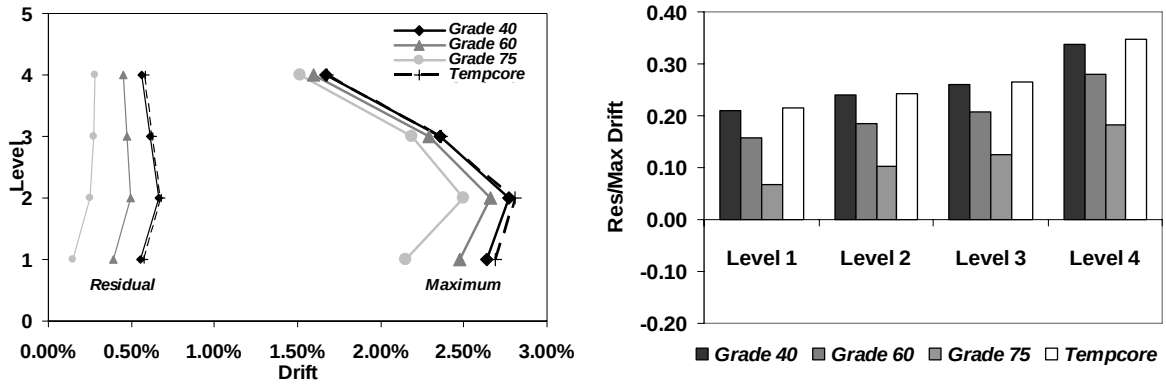
Figure 2: Normalised (a) reinforcing steel stress-strain curves (b) full M-ϕ response (c) bilinear approximation to M-ϕ response

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.

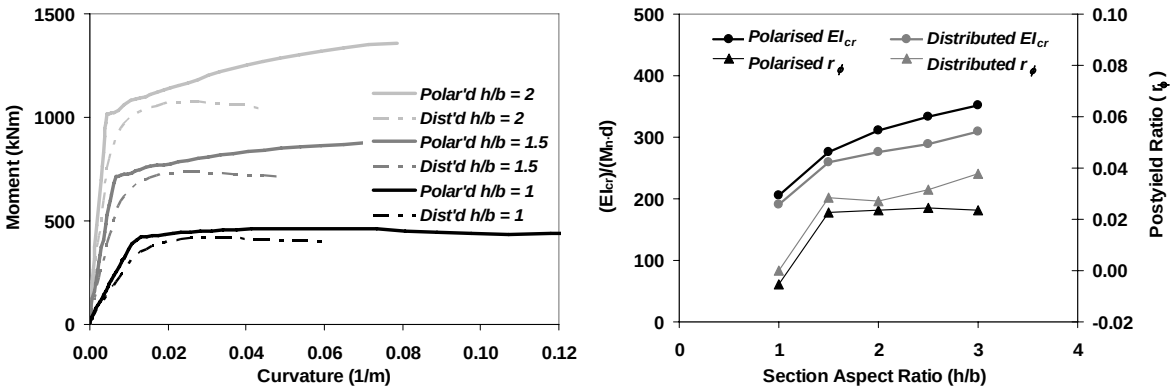
4.2 Effect of Section Design on Post-Yield Stiffness

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

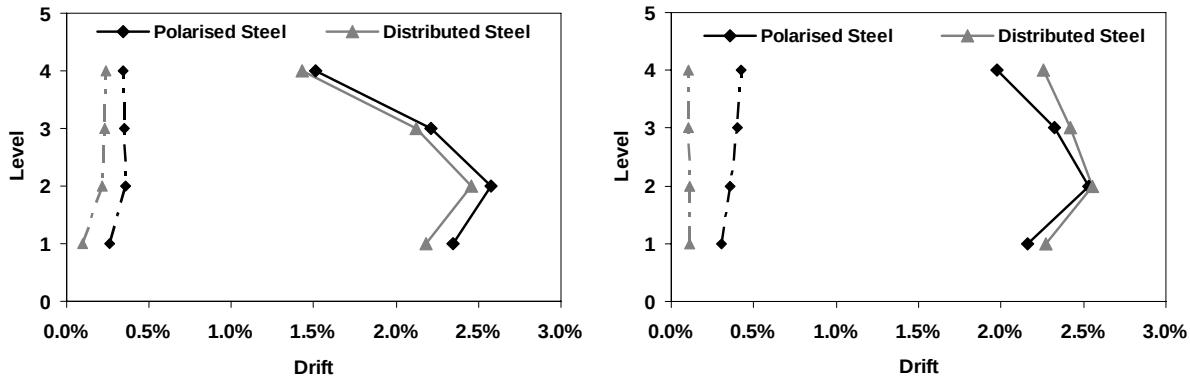
have a consistently higher post-yield stiffness ratio (as defined from the standard bilinearisation approach described earlier). Note that Figure 4b highlights the slight increases in post-yield ratio with aspect ratio (and decreasing longitudinal steel ratio) due to the higher steel stresses developed.



(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.

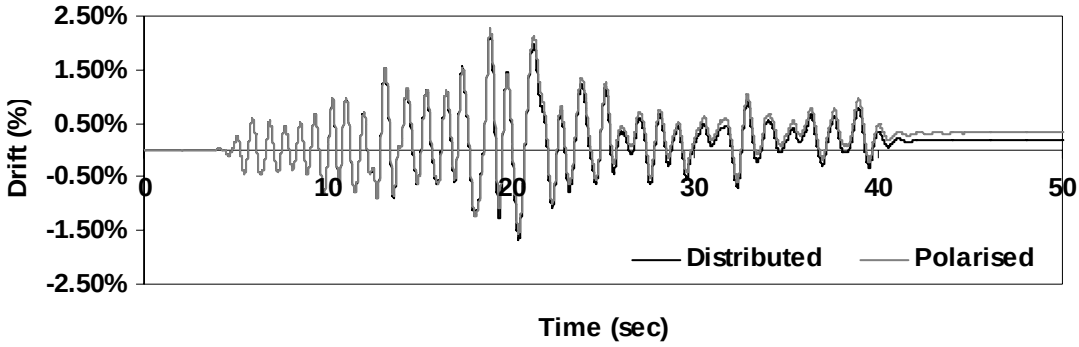


Figure 6: Time-history of 4-storey RC MRF at effective height (at the centre-of-force) comparing polarised and distributed beam reinforcement layouts (using Ruaumoko 2D).

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]:

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

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 = 2465kN V_1 = 2335kN					Secondary MRF V_2 = 130kN	
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;

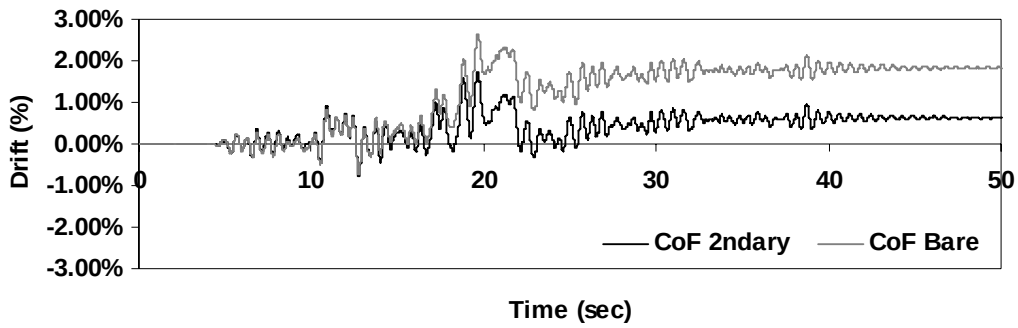


Figure 7: Time-history response at the centre-of-force (CoF) of the initial 'Bare' BRBF with no secondary frame and the comparative response when a secondary frame is included.

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.

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