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Development and validation of a metallic haunch seismic retrofit solution for existing under-designed RC frame buildings

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SUMMARY

The feasibility and efficiency of a seismic retrofit solution for existing reinforced concrete frame systems, designed before the introduction of modern seismic-oriented design codes in the mid 1970s, is conceptually presented and experimentally investigated. A diagonal metallic haunch system is introduced at the beam–column connections to protect the joint panel zone from extensive damage and brittle shear mechanisms, while inverting the hierarchy of strength within the beam–column subassemblies and forming a plastic hinge in the beam. A complete step-by-step design procedure is suggested for the proposed retrofit strategy to achieve the desired reversal of strength hierarchy. Analytical formulations of the internal force flow at the beam–column-joint level are derived for the retrofitted joints. The study is particularly focused on exterior beam–column joints, since it is recognized that they are the most vulnerable, due to their lack of a reliable joint shear transfer mechanism. Results from an experimental program carried out to validate the concept and the design procedure are also presented. The program consisted of quasi-static cyclic tests on four exterior, $\frac{2}{3}$ scaled, beam–column joint subassemblies, typical of pre-1970 construction practice using plain round bars with end-hooks, with limited joint transverse reinforcement and detailed without capacity design considerations. The first (control specimen) emulated the as-built connection while the three others incorporated the proposed retrofitted configurations. The experimental results demonstrated the effectiveness of the proposed solution for upgrading non-seismically designed RC frames and also confirmed the applicability of the proposed design procedure and of the analytical derivations.

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KEY WORDS: existing buildings; seismic retrofit; under-designed frames; haunch; shear hinge; beam–column joints

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1. INTRODUCTION

Recent experimental investigations on the seismic performance of existing reinforced concrete frame buildings, designed for gravity loads only, as typically found in seismic prone countries before the introduction of adequate seismic code provisions in the mid-1970s, have confirmed the expected inherent weaknesses of these systems [1–7], that have been observed in past earthquake events. Because of the poor detailing of the reinforcement, the absence of capacity design philosophy and the use of plain round reinforcing bars, undesirable brittle failure mechanisms are observed at either the local level (i.e. shear failures in joints, beam or column members) or globally in the structure (i.e. soft-storey mechanisms). The beam–column joint panel region is of particular interest in such systems, as it is likely to be the critical and possibly the *weakest link* according to capacity design or hierarchy of strength considerations. Joint damage and failure can in fact lead to severe deterioration of the overall lateral load carrying capacity of the structure and even result in total collapse. Appropriate retrofit strategies, capable of providing adequate protection to the joint region while modifying the strength hierarchy between the different components of the beam–column connections, according to a capacity design philosophy, are thus required for improving the seismic response of such structures.

Several strengthening/retrofit solutions have been studied in the past and have been adopted in practical applications, ranging from conventional techniques (i.e. braces, jacketing or infills [8]) to more recent approaches including base isolation, supplemental damping devices or advanced non-metallic materials as fibre reinforced polymers (FRP) [9], or shape memory alloys (SMAs) [10]. Most of these retrofit techniques have evolved into viable upgrades of these seismically deficient structures. However, issues of cost, invasiveness, and practical implementation still remain the most challenging aspects of retrofitting non-seismically designed RC frames.

In this contribution, the feasibility and efficiency of a simple, low-invasive and cost-effective retrofit solution, which relies on diagonal metallic haunches installed locally at the beam–column joints to protect the panel zone and to force a more desirable hierarchy of strength, is conceptually presented, implemented and experimentally validated. Experimental results from quasi-static cyclic tests on four exterior beam–column subassemblies, at a $\frac{2}{3}$ scale, comprising of one as-built specimen and three retrofitted solutions, confirmed the efficiency and reliability of the proposed retrofit solution and of the proposed design methodology.

2. SEISMIC BEHAVIOUR OF POORLY DESIGNED RC FRAMES

2.1. *Typical structural deficiencies of pre-1970s frame buildings*

As it has been widely reported in the literature [1, 7], typical structural deficiencies of existing reinforced concrete frame systems are most often related to:

(a) Inadequate confining effects in the potential plastic hinge regions.
(b) Insufficient amount, if any, of transverse reinforcement in the joint regions.
(c) Low amount (nominal) of longitudinal and transverse reinforcement in columns and beams.
(d) Inadequate anchorage detailing (including end-hook solutions), for both longitudinal and transverse reinforcement.

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(e) Lapped splices of column reinforcement just above the floor level.
(f) Lower quality of materials (concrete and steel) when compared to current practice:

- plain round (smooth) bars for both longitudinal and transverse reinforcement;
- low-strength concrete.

The main variations between construction practices in different seismic-prone countries are related to the percentage of column longitudinal reinforcement, which strongly affects the beam-to-column moment capacity ratio (increasing the tendency of developing soft-storey mechanisms), the different anchorage details in lap splice regions or within joint regions, as well as the minimum spacing of the transverse reinforcement in beams and columns, affecting the shear capacity of the framing elements. Other observed differences are related to the use of shallow and wide beams (typical in Mediterranean countries, abandoned in the late 1950s in NZ) instead of deeper beams, as well as to the type of slab (e.g. cast-in-situ versus lightweight hollow clay bricks).

2.2. *Vulnerability of the panel zone region*

As part of an extensive recent experimental and analytical research program on the seismic vulnerability of existing reinforced concrete frame buildings designed for gravity loads only, as typically found in Mediterranean countries before the introduction of seismic-oriented codes in the mid-1970s, a series of quasi-static cyclic tests on $\frac{2}{3}$ scaled beam–column joint subassemblies (interior and exterior) as well as on two three-storey three-bay frame systems, prior to and after retrofit using FRP solutions, have been carried out at the University of Pavia [6, 7, 11, 12]. These tests confirmed and further highlighted the vulnerability of the panel zone region. Due to the absence of capacity design considerations and the peculiar combination of plain round bars and end-hook anchorages, particularly brittle joint shear damage mechanisms were observed and are expected in exterior beam–column joints (see Figure 1), with the development of a joint shear mechanism before the occurrence of any flexural hinging in the beam (or more likely in the column). At a global system level, these local damage mechanisms (including column hinging in interior beam–column joint) could result in rapid strength deterioration of the lateral load carrying system, low levels of displacement ductility as well as soft-storey mechanisms.

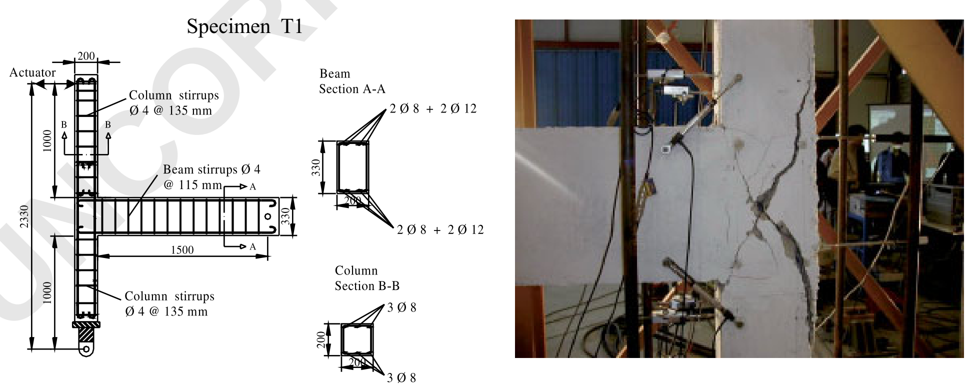

Figure 1. Typical geometry and reinforcement details of a pre-1970 beam column and experimental joint shear failure mechanism [6].

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Different damage or failure modes are expected to occur in non-seismically designed beam–column joints [6, 13] depending on the typology (exterior or interior joint) and on the detailing (i.e. amount, if any, of transverse reinforcement in the joint; use of plain round or deformed bars; bar anchorage detailing). In absence of transverse reinforcement in the joint region, the post-cracking behaviour depends solely on the efficiency of the compression strut mechanism to transfer the shear within the joint. Thus, while rapid joint strength degradation after joint diagonal cracking is expected in exterior joints, a hardening behaviour after first diagonal cracking is usually observed for interior joints [13]. On the other hand, the concentration of shear deformation in the joint region, through the activation of a shear hinge mechanism [13], can reduce the deformation demand on adjacent structural members, postponing the occurrence of undesirable soft-storey mechanisms which can lead to the collapse of the entire structure. A critical discussion on the effects of damage and failure of beam–column joint (panel zone) in the seismic assessment of frame systems can be found in Reference [11].

With the original intent to develop a comprehensive database for the characterization and modelling of the joint panel zone shear damage mechanism, further experimental investigations have been carried out and are ongoing at the University of Canterbury in New Zealand. These tests include quasi-static uni- and bi-directional loading on exterior beam–column joint subassemblies with different structural details (either plain round or deformed bars, without stirrups or with a single horizontal stirrup in the joint, deep beam or shallow–wide beam) [14]. One of the tested pre-1970s as-built solutions has been selected in this study as a benchmark or control specimen for the retrofit solution and is discussed in more detail in the following paragraphs. The extensive analytical and numerical modelling of existing non-seismically designed RC frames is beyond the scope of this paper. The readers are referred to References [13, 15, 16] for recent overviews of different analytical procedures reported in the literature.

3. RETROFIT SOLUTION USING A DIAGONAL HAUNCH SYSTEM

3.1. Conceptual challenges and retrofit strategy

A retrofit solution for existing under-designed RC frame systems is herein proposed as an extension of the haunch retrofit solution developed for steel moment resisting frames following the significant number of weld fractures observed after the Northridge earthquake [17–19]. It is worth underlining that the main scope of the haunch retrofit solution, as originally proposed for moment resisting steel frames, was primarily to relocate the plastic hinge away from the welded connection to protect the welds from premature cracking. These moment-resisting steel frame systems were already typically designed according to capacity design considerations, thus prone to develop hinging in the beams (weak-beam/strong column system) if the welds were adequately protected.

When applying this retrofit solution to existing reinforced concrete frame buildings designed without capacity design considerations, additional challenges arise since it is not guaranteed that the desired beam flexural hinging mechanism will develop even if the panel zone is adequately protected. The primary aim of the proposed seismic retrofit strategy is thus to eliminate the damage in the beam-to-column panel zones while enhancing the global response of non-seismically designed RC frames by reversing the hierarchy of strength. Local haunch type elements, as illustrated in Figure 2, are introduced in the vicinity of the beam-to-column connections to protect the panel zone region from excessive damage by re-directing the stress-flow around the joint region

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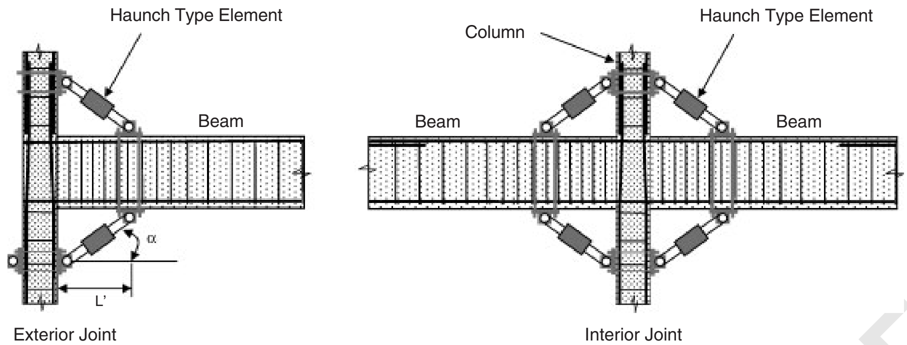

Figure 2. Proposed haunch retrofit configuration for exterior and interior joints.

and forcing the development of a relocated plastic hinge in the beam. The proposed steel haunch is used for both exterior and interior beam-to-column joints. The design of the retrofit consists of properly selecting the geometry (distance from the column interface, L', and angle α) as well as the axial stiffness K_d of the haunch elements, such that the moment developed in the beam at the face of the column is controlled, thus protecting the joint panel zone from undesirable brittle failure mechanisms. Furthermore, the design must also assure the reversal of the strength hierarchy by forcing a flexural plastic hinge in the beam close to the location where the haunch is connected. It is intended, through capacity design considerations, that shear failure mechanisms be avoided in both beams and columns when the relocated flexural hinges are formed in the beams.

As proposed by Christopoulos and Filiatrault [19], based on numerical investigations on steel moment resisting frames, the haunch type elements can also be designed as stiffening elements with sufficient strength to remain elastic under the applied loads, or as passive elasto-plastic devices which rely either on hysteretic yielding or on friction type elements to provide supplemental damping to the system. Experimental implementation of both elastic and dissipating (through yielding) haunch solutions are presented and discussed in the next paragraphs, based on the tests that were carried out on existing pre-1970 designed RC beam–column joints.

3.2. Effect of haunch elements on internal forces at beam–column joint subassemblies

When haunch type elements are introduced at a distance L' from the beam–column interface and connected at an angle α above and below the beam (see Figure 2), the internal forces of the beam–column assembly are significantly altered. Figures 3 and 4 and the paragraph below illustrate the effects of the haunch retrofit solution on moment and shear force diagrams in an exterior beam–column joint subassembly subjected to lateral loads (with inflexion points assumed at mid-height of the column and mid-span of the beam). If designed adequately, the presence of the two haunches can significantly reduce the beam and column moments at the joint panel zone interface. The maximum moment in the beam and in the column is relocated away from the original critical sections to the points where the haunches are connected. In particular, as discussed in the following paragraphs, the migration of the maximum moment in the beam at a distance L' from the face of the column can be exploited to force a plastic hinge in the beam. It is of interest to also note that a similar, though less efficient, reduction of internal forces at the joint level can be achieved by using a single haunch element, introduced only below (or above) the beam, in order

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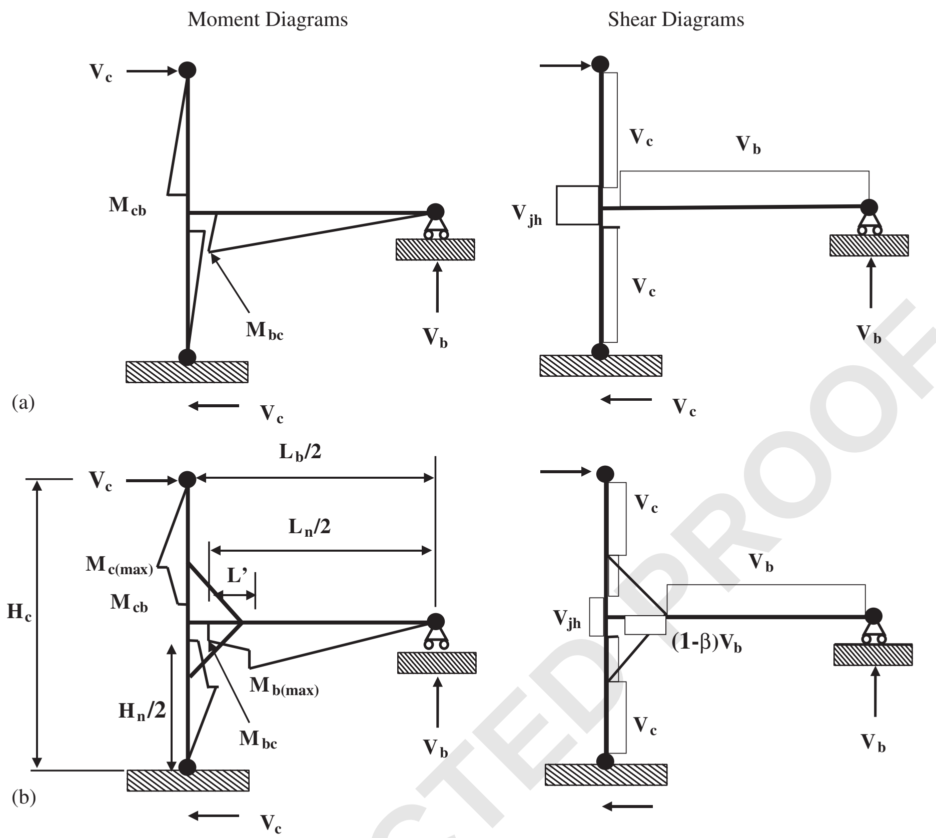

Figure 3. Moment and shear diagrams of exterior joint: (a) as-built solution; and (b) after retrofitting with two diagonal haunches.

to meet architectural requirement with an even less invasive retrofit intervention. The efficiency of the haunch intervention in modifying the internal shears and moments in the beams and columns is dependent on the selection of the three haunch design parameters L', α and the axial stiffness of the haunch K_d. As illustrated in Figure 4, once the shear in the beam between the point of connection of the haunch (to the beam) and the face of the column βV_{b} is determined, the moment and shear diagrams from the point of inflexion to the face of the column are known. The factor β is assumed to be known in the following expressions. The derivation of the factor β based on displacement compatibility is discussed in detail in a subsequent paragraph. The beam moment at the column interface, M_{bc}, is thus given by

$$M_{\mathrm{bc}} = M_{\mathrm{b(max)}} - \Delta M_H - (1-\beta) V_{\mathrm{b}} L' \tag{1}$$

where $\Delta M_H = [\beta V_{\mathrm{b}}(d_{\mathrm{b}}/2)]/\tan\alpha$ is the concentrated moment reduction at a distance L' from the face of the column (haunch location) due to the offset of the beam centreline from the point where the haunches are connected to the beam (see Figure 2) and d_{b} is the depth of the beam. If the haunch-beam connection is done at mid-depth of the beam, this localized moment reduction would

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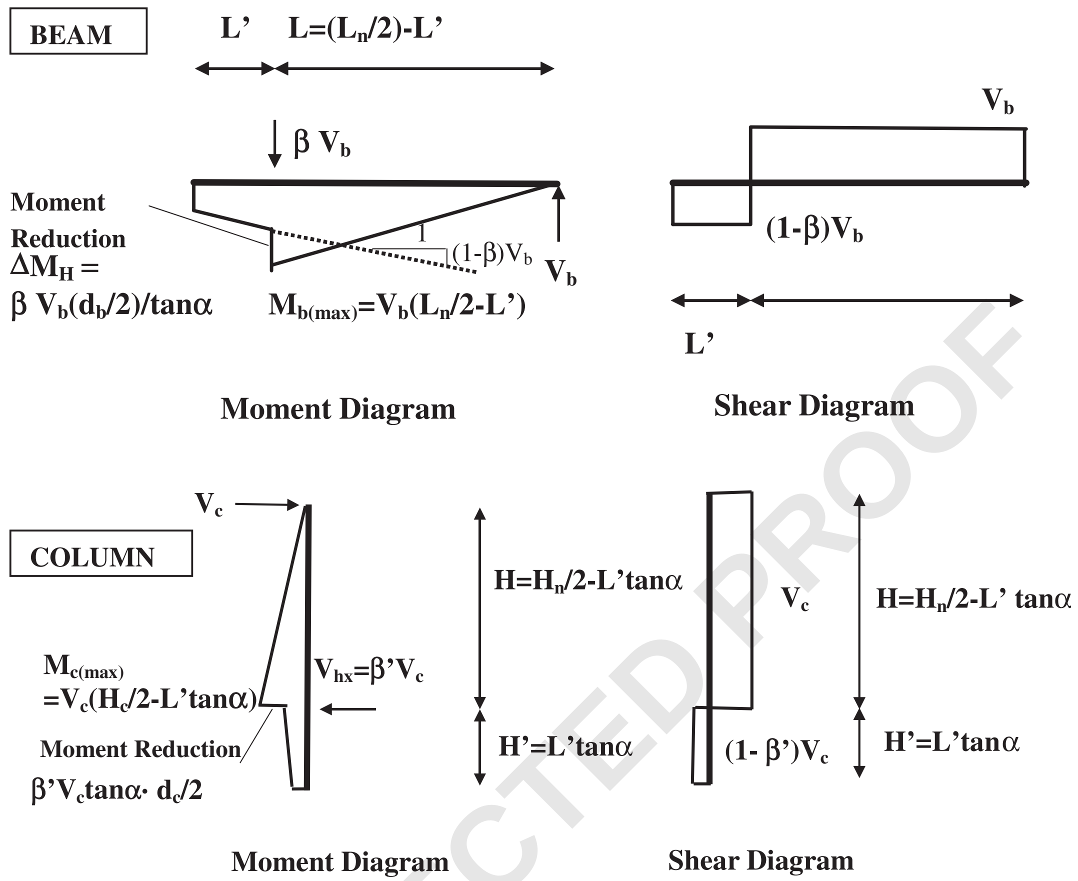

Figure 4. Moment and shear diagrams in the beam and column after retrofitting with two diagonal haunches.

not be present and is therefore not considered in the derivation of a general design procedure. As a result:

$$M_{bc} = M_{b(max)}\left[1 - \frac{\beta d_b}{2L\tan\alpha} + \frac{(1-\beta)L'}{L}\right] \tag{2}$$

where $L = (L_n/2) - L'$ is the distance from the middle span of the beam to the location of the haunch with L_n being the net beam span length (from face to face of columns).

Similarly, the reduction of the column moment at the beam interface can be expressed as

$$M_{cb} = M_{c(max)}\left[1 - \frac{\beta' d_c\tan\alpha}{2H} + \frac{(1-\beta')L'\tan\alpha}{H}\right] \tag{3}$$

where $H = (H_n/2) - L'\tan\alpha$ is the distance between the point of contraflexure at mid-height of the column and the haunch location, d_c is the depth of the column and $\beta' = \beta \cdot (H_c/L_b\tan\alpha)$ with H_c the total interstorey height (from centreline to centreline of beams), and L_b the total beam

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span length (from centreline-to-centreline of columns). The maximum column moment developed at the level of the haunch connection is therefore:

$$M_{\mathrm{c(max)}} = M_{\mathrm{b(max)}} \frac{(H_{\mathrm{c}}/2 - d_{\mathrm{b}}/2 - L' \tan\alpha)(1 + (d_{\mathrm{c}} + 2L')/(L_n - 2L'))}{H_{\mathrm{c}}} \tag{4a}$$

or, alternatively

$$M_{\mathrm{c(max)}} = M_{\mathrm{b(max)}} \frac{L_{\mathrm{b}} H}{L H_{\mathrm{c}}} \tag{4b}$$

4. DESIGN PHILOSOPHY AND PROCEDURE

4.1. Protection of the panel zone and hierarchy of strength

As discussed in the previous paragraphs, the proposed retrofit strategy aims to protect the panel zone region while forcing a plastic flexural hinge in the beam at the location of the haunch connection. Capacity design considerations are derived to guarantee that a proper hierarchy of strength is developed. When designing the haunch solution (choice of L', α and K_d), control of the actual hierarchy of strength can be obtained by imposing that the equivalent interstorey shear corresponding to the development of a relocated plastic hinge in the beam at a distance L' from the face of the column is lower than those corresponding to undesirable critical mechanisms. These mechanisms, from the least severe to the most severe on the overall integrity of the structure are: (i) column hinging, (ii) joint shear failure, (iii) beam shear failure, and (iv) column shear failure. The global target hierarchy of strength of the whole design can be thus summarized as

$$\overline{V}_{\mathrm{c,beam\text{-}hinge}} \leqslant \Phi_1 \overline{V}_{\mathrm{c,col\text{-}hinge}} \leqslant \Phi_2 \overline{V}_{\mathrm{c,joint}} \leqslant \Phi_3 \overline{V}_{\mathrm{c,col\text{-}shear}} \leqslant \Phi_4 \overline{V}_{\mathrm{c,beam\text{-}shear}} \tag{5}$$

where Φ_i are safety factors separating two subsequent mechanisms.

4.2. Step-by-step haunch design procedure

The proposed design procedure consists of iterations on the properties defining the haunch system and is presented in the following steps:

Step 1: *Preliminary choice of haunch properties*. A preliminary selection of the properties of the haunches L', α and K_d is first required. From a practical point of view, especially for frames with weak column problems, starting with higher values of α will lead to a more effective retrofit and a quicker convergence to feasible solutions. Furthermore, to reduce the invasiveness of the retrofit strategy, the lowest possible value of L' is preferred. The value of K_d is also limited by the choice of the haunch element sections and materials. More than one combination of the haunch properties may satisfy these requirements and it may be useful to investigate a number of possible combinations.

Step 2: *Definition of acceptable damage/limit states in the joint*. The starting point of the design scheme is to define the acceptable stress level in the joint, based on principle tensile stresses p_{t} or principle compression stresses, p_{c}, in the case of interior joints. As noted in the literature [20], principal stresses are more reliable than nominal shear stresses in capturing the response of joints, since they consider the actual stress state of the joint given by a combination of the nominal shear force, V_{jh} (or corresponding stress, v_{jh}) with the column axial load (N). The

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variation of column axial load during the lateral sway of a frame building can be quite significant, particularly in exterior beam–column joints, and should be properly accounted for when defining the actual stress level. Strength degradation curves for existing beam–column joints based on principle tensile/compression stresses as a function of the joint shear deformation have been proposed in the literature [3, 6, 20]. Values of the principal tensile stress of approximately $p_t = 0.2\sqrt{f_c'}$, f_c' being the concrete compression strength, have been for example proposed by Pampanin *et al.* [6] to correspond to first cracking of exterior beam–column joints with plain round bars and end hooks. In a retrofitted configuration, the nominal (horizontal) shear force in the joint V_{jh} can be expressed as a function of the moment in the beam at the joint interface:

$$V_{jh} = \frac{M_{bc}}{jd_b} - (V_c - V_{hx}) \tag{6}$$

where jd_b is the internal lever arm in the beam section between the tension and compression sides and $(V_c - V_{hx})$ or, $(1-\beta')V_c$ is the actual shear value in the column at the panel zone interface (see Figure 4) with $V_{hx} = \beta' V_c$ being the horizontal component of the haunch force acting on the column expressed by

$$V_{hx} = \frac{2\beta V_c H_c}{2\tan\alpha(L_n + d_c)} \tag{7}$$

is the horizontal shear force introduced into the column from the haunch. By substituting Equations (2) and (7) into Equation (6) and solving for V_c, the interstorey shear corresponding to the occurrence of a defined level of shear or principle tensile stress demand (i.e. damage) in the joint, herein referred to as $\overline{V}_{c,joint}$, is given by

$$\overline{V}_{c,joint} = \frac{A_e\sqrt{p_t^2 - (p_t N/A_g)}}{\left(1 - \dfrac{\beta H_c}{(L_n + d_c)\tan\alpha} - \dfrac{H_c(L_n - 2L')}{jd_b(L_n + d_c)}\left(1 - \dfrac{\beta d_b}{2L\tan\alpha} + \dfrac{(1-\beta)L'}{L}\right)\right)} \tag{8}$$

where the numerator also represents the joint nominal shear force $V_{jh} = A_e\sqrt{p_t^2 - (p_t N/A_g)}$ (A_g being the column gross section and A_e the effective joint area).

Alternatively, a relationship between a given limit state in terms of joint shear and the beam moment at the joint interface, $\overline{M}_{bc}$ (or $\overline{M}_j$), can be written as

$$\overline{V}_{c,joint} = \frac{\overline{M}_{bc}/jd_b}{\left(2 - \dfrac{2\beta H_c}{(L_n + d_c)\tan\alpha} - \dfrac{H_c(L_n - 2L')}{jd_b(L_n + d_c)}\left(1 - \dfrac{\beta d_b}{2L\tan\alpha} + \dfrac{(1-\beta)L'}{L}\right)\right)} \tag{9}$$

Step 3: *Force the development of a plastic hinge in the beam*. The beam will hinge when the maximum moment at the haunch connection $M_{b(max)}$ reaches the yielding moment capacity of the beam, M_{by}. The corresponding equivalent moment in the beam at the joint interface M_{bc} is obtained simply by substituting $M_{b(max)} = \overline{M}_{by}$ into Equation (2). The equivalent interstorey shear (subassembly lateral force) corresponding to the development of a plastic hinge in the beam is therefore given by

$$\overline{V}_{c,beam\text{-}hinge} = \overline{M}_{by}\frac{(1 + (d_c + 2L')/(L_n - 2L'))}{H_c} \tag{10}$$

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In order to guarantee that the development of a plastic hinge in the beam occurs before a predefined level of damage is suffered by the joint, the following capacity design equation must be met:

$$\bar{V}_{\mathrm{c,beam\text{-}hinge}} \leqslant \Phi_1 \cdot \Phi_2 \bar{V}_{\mathrm{c,joint}} \tag{11}$$

Step 4: *Check column is not hinging prior to beam or joint*. It is important to recall that the development of the plastic hinge in the beam should also occur before the formation of a plastic hinge in the column at the location of the haunch (maximum moment $M_{\mathrm{c(max)}}$). This hierarchy of strength can thus be guaranteed by ensuring that:

$$\overline{V}_{\mathrm{c,beam\text{-}hinge}} < \Phi_1 \overline{V}_{\mathrm{c,column\text{-}hinge}} < \Phi_2 \overline{V}_{\mathrm{c,joint}} \tag{12}$$

where $V_{\mathrm{c,col\text{-}hinge}}$ is the equivalent interstorey shear (subassembly lateral force) corresponding to the development of a plastic hinge in the column and given by:

$$\overline{V}_{\mathrm{c,col\text{-}hinge}} = \frac{\overline{M}_{\mathrm{c}y}}{(H_{\mathrm{c}}/2 - d_{\mathrm{b}}/2 - L' \tan\alpha)} \tag{13}$$

where $\overline{M}_{\mathrm{c}y}$ is the yield moment of the column.

Step 5: *Final check of shear capacity in members and control of global hierarchy of strength and sequence of events*. The final check to meet the desired hierarchy of strength as expressed in Equation (5) is achieved once the shear capacity of beam and column are checked against the shear demand that develops in each of these elements in the retrofitted configuration. It is worth recalling that typical code provisions for either design or assessment (i.e. Reference [21]) suggest to neglect the concrete contribution when evaluating the shear capacity within a plastic hinge region as for example in beams. Furthermore, when plain round bars are used, as typical of pre-1970s RC buildings, a main flexural crack is expected to develop directly at the critical interface and progressively widen due the premature loss of bond properties and slip of the bars, as confirmed by a large number of experimental investigations carried out by numerous researchers on effects of plain round bars (e.g. Reference [22]). In such conditions, different from the extensive cracking patterns typically observed when a plastic hinge forms over a certain length, the efficiency of the beam transverse reinforcement in carrying the internal shear forces can be substantially impaired. In such cases, dowel effects of the longitudinal bars are relied upon to transfer shear. To improve the shear transfer at this critical location, simple metallic corbels or shear keys can be incorporated into the system by extending the connection plates of the haunch at the beam level a few centimeters towards the middle of the beam. In the validation experiments presented in the following paragraphs, no shear keys (corbels) were used to demonstrate that the dowel effect is sufficient in transferring the beam shear.

Iteration process: The iteration process consists of redefining the haunch properties (step 1) every time either one of steps 2–4 are not satisfied. The above steps can easily be programmed into a spreadsheet for rapid use by designers. If certain parameters of the system make it unfeasible for all the steps to be met, alternative solutions can be combined with the proposed haunch to meet the design requirements. Due to wide differences in the spacing of the transverse reinforcement in different regions of the world (due to different minimum requirements in codes or by-laws in the 1950–1970s, as well as different construction practices from country to country), in some cases the shear capacity of an existing under-designed column or beam (at a section away from the column interface) is not sufficient to guarantee a proper inversion of the hierarchy of strength with the formation of the desired flexural hinge in the beam. As further discussed in Section 5.6,

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although columns were originally not designed for the increased shear demands associated with the introduction of the haunch elements their reserve shear capacity is usually adequate to carry this additional shear. In such cases, a simple hybrid retrofit solution could be adopted whereby the haunch retrofit intervention is combined with other means of protecting the structural elements from shear failure. Composite materials (FRP) in the form of strips, sheets, or rods as well as other forms of local jacketing (i.e. steel plates) could be for example adopted. Such additional interventions could also improve the flexural strength of the column, while still allowing for an intervention that is overall less invasive than most of traditional retrofits where retrofit of the joint panel zone causes major disruption of the floor system around the column. Similarly, vertical post-tensioned bars or externally mounted surface could be adopted to increase the shear (as well as flexural) capacity of the column (as well as of the joint).

4.3. Evaluation of the β-factor to account for deformation compatibility

As illustrated in Figure 4 and in the aforementioned equations, the β-factor is a critical design parameter. Once evaluated, it allows for the complete definition of moments and shears in beams and columns and is therefore critical in the definition and design of the proposed retrofit solution.

The value of the β-factor can be determined by writing deformation compatibility equations between the axial deformation of the haunch and the local deformations of beams and columns where the haunch is connected. The complete formulation of such an equation involves axial, flexural and shear deformations in both beams and columns as well as panel zone elastic shear deformations. However, depending on the relative stiffness of elements and the relative contribution of these deformations to the total local deformation, simpler equations neglecting some of these contributions can be derived. A first derivation of the β-factor has been proposed by Yu *et al.* [17] in the formulation of a (single) haunch retrofit solution for steel frame buildings, which for simplicity accounts only for the beam flexural deformations. Adapting this equation to the configuration proposed in this paper (shown in Figures 2 and 3), the following expression can be derived:

$$\beta = L'\left(\frac{-6Ld_{\mathrm{b}}\sin\alpha\cos\alpha - 3L'd_{\mathrm{b}}\sin\alpha\cos\alpha - 6L'L + 6L'L\cos^2\alpha - 4L'^2 + 4L'^2\cos^2\alpha}{-3\cos^2\alpha\, d_{\mathrm{b}}^2 L' - 6d_{\mathrm{b}}L'^2\sin\alpha\cos\alpha - 4L'^3\cos^2\alpha - 12EI_{\mathrm{b}}/(2K_d)}\right) \tag{14}$$

where I_{b} is the moment of inertia of the beam and K_d is the axial stiffness of one haunch element. A simpler expression can be obtained by substituting $a = L'$ and $b = L'\tan\alpha$:

$$\beta = \left(\frac{b}{a}\right)\cdot\frac{6Ld_{\mathrm{b}} + 3ad_{\mathrm{b}} + 6bL + 4ab}{3d_{\mathrm{b}}^2 + 6bd_{\mathrm{b}} + 4b^2 + (12EI_{\mathrm{b}}/2K_d a\cos^2\alpha)} \tag{15}$$

Considering the moment diagram presented in Figure 4, it can be seen that values of β greater than 1 are desirable for a more efficient protection of the beam-to-column joint. For given properties of beam and column sections, a number of combinations of L', α and K_d are possible such that the final design solution can limit the invasiveness of the added haunch elements while providing the necessary or targeted upgrade to the system. As a general rule, larger values of β reduce the effect of the haunches on the maximum moments in the columns and are therefore preferred for cases where weak-column behaviour is expected. However, in order to avoid excessive increase of shear demands in the beam and column elements, β should generally not exceed a value of 2.

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4.4. Refinement of deformation compatibility equation to account for column deformation

As will be further discussed in the following paragraphs, a conservative design was followed in the first phase of the experimental project using the original formulation of the β-factor which neglects column and joint deformability (Equations (14) or (15)). The first experimental results (presented in subsequent paragraphs) highlighted the inaccuracy of such a formulation which neglects deformations in the column. Although this equation was adequate for steel moment resisting frames where column deformations are smaller, for existing pre-1970s frame systems where columns were designed only considering gravity loads, these deformations are significant.

Two more refined versions of the β-formulation were therefore derived to also account for: (i) the column flexibility and (ii) the column and joint flexibility. Equation (16) shows the formulation of the β-factor when including column flexibility in addition to beam flexibility. The additional terms (when compared to Equation (15)) related to the column flexibility are highlighted in boxes:

$$\beta=\left(\frac{b}{a}\right)\cdot\frac{6Ld_{\rm b}+3ad_{\rm b}+6bL+4ab+\boxed{\dfrac{2I_{\rm b}L_{\rm b}b^3}{I_{\rm c}aH_{\rm c}}+\dfrac{3I_{\rm b}HL_{\rm b}b^2}{I_{\rm c}aH_{\rm c}}+\dfrac{3I_{\rm b}d_{\rm c}L_{\rm b}b^3}{2I_{\rm c}a^2H_{\rm c}}+\dfrac{3I_{\rm b}d_{\rm c}HL_{\rm b}b^2}{I_{\rm c}a^2H_{\rm c}}}}{3d_{\rm b}^2+6bd_{\rm b}+4b^2+\dfrac{12EI_{\rm b}}{(2K_d a\cos^2\alpha)}+\boxed{\dfrac{6I_{\rm b}b^2}{a^2A_{\rm c}}+\dfrac{2I_{\rm b}b^3}{I_{\rm c}a}+\dfrac{3I_{\rm b}d_{\rm c}b^2}{I_{\rm c}a^2}+\dfrac{3I_{\rm b}d_{\rm c}^2b^3}{2I_{\rm c}a^3}}} \tag{16}$$

where $A_{\rm c}$ and $I_{\rm c}$ are, respectively, the gross-section area and the moment of inertia of the column. When the joint flexibility is also included, the complete formulation of the β-factor is given by Equation (17), where the separate contribution from the beam (terms with no brackets), column (terms in [ ] brackets) and joint deformations (terms in { } parentheses) are indicated in the numerator and denominator:

$$\beta=\left(\frac{b}{a}\right)\cdot\frac{\begin{array}{c}6Ld_{\rm b}+3ad_{\rm b}+6bL+4ab+\boxed{\dfrac{2I_{\rm b}L_{\rm b}b^3}{I_{\rm c}aH_{\rm c}}+\dfrac{3I_{\rm b}HL_{\rm b}b^2}{I_{\rm c}aH_{\rm c}}+\dfrac{3I_{\rm b}d_{\rm c}L_{\rm b}b^3}{2I_{\rm c}a^2H_{\rm c}}+\dfrac{3I_{\rm b}d_{\rm c}HL_{\rm b}b^2}{I_{\rm c}a^2H_{\rm c}}}\\ +\left\{\dfrac{12EI_{\rm b}}{K_ja}\left(b+\dfrac{d_{\rm b}}{2}\right)\left(a+\dfrac{d_{\rm c}}{2}+L\right)\right\}\end{array}}{\begin{array}{c}3d_{\rm b}^2+6bd_{\rm b}+4b^2+\dfrac{12EI_{\rm b}}{(2K_d a\cos^2\alpha)}+\boxed{\dfrac{6I_{\rm b}b^2}{a^2A_{\rm c}}+\dfrac{2I_{\rm b}b^3}{I_{\rm c}a}+\dfrac{3I_{\rm b}d_{\rm c}b^2}{I_{\rm c}a^2}+\dfrac{3I_{\rm b}d_{\rm c}^2b^3}{2I_{\rm c}a^3}}\\ +\left\{\dfrac{12EI_{\rm b}b}{K_ja}\left(b+\dfrac{d_{\rm c}}{2}\right)\left(1+\dfrac{d_{\rm b}}{2b}+\dfrac{d_{\rm c}}{2a}\right)\right\}\end{array}} \tag{17}$$

Details on the full derivation of the β-factor to account for beam, column and joint flexibilities can be found in Reference [23]. A comparison between the different formulations of the β-factor (Equations (15)–(17)) are shown in Figure 5. Theoretical predictions using Equations (15)–(17) are shown in the form of design charts and compared with numerical results from SAP2000, where elastic frame models have been used to represent the retrofitted beam–column joint. It is interesting to note that the values of β evaluated from a beam-only-flexibility tend to be lower (up to 15–20%)

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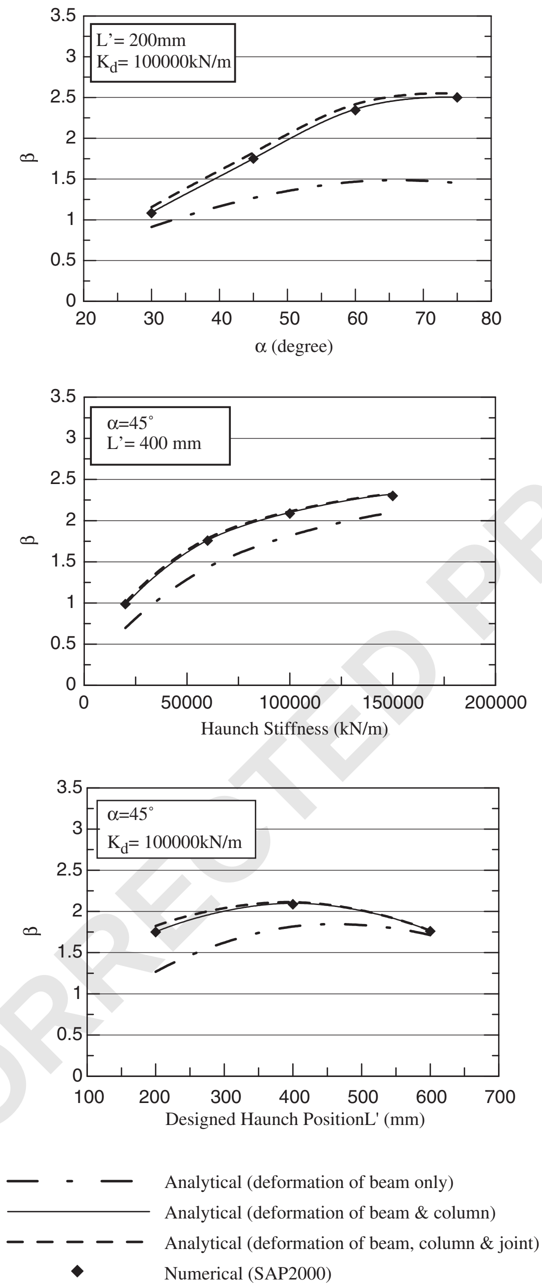

Figure 5. Comparison between alternative formulations of the haunch β-factor when accounting for deformation contribution of beam, column and/or joint elements.

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than the corresponding values calculated including the column deformability, thus underestimating the efficiency of the added haunch in protecting the panel zone (conservative) as well as the shear forces developed in the structural elements (unconservative). The values obtained when including the column or the column and joint deformability were very close to the values obtained using the SAP2000 model. It can also be seen in Figure 5 that when the column deformations are included, neglecting the joint deformations has a small impact on the predicted values of the β-factor.

4.5. Multi-level retrofit strategies to control the global behaviour

The retrofit strategy outlined thus far has been focused on the local (subassembly) level, assuming that in a real design situation, all interior and exterior joints would be retrofitted. According to a multi-level retrofit strategy approach [12, 24], two levels of retrofits can be adopted when considering the global response of the frame: (i) a *complete retrofit* would consist of a full upgrade by protecting all joint panel zones and developing plastic hinges in beams while columns are protected according to capacity design principles, and (ii) a *partial retrofit* would consist of protecting exterior joints (which are the most vulnerable), forming plastic hinges in beams framing into exterior columns, while allowing for column hinging and minor joint cracking in interior joints due to their more stable behaviour with hardening after first cracking.

4.6. Preliminary numerical investigations and feasibility studies

Preliminary numerical investigations and feasibility studies on the efficiency of the proposed *complete* and *partial* retrofit solutions were presented by Pampanin and Christopoulos [24]. It is worth recalling that when assessing the seismic performance of existing pre-1970s buildings) under-designed or designed-for-gravity-only buildings via numerical/analytical investigations, an adequate modelling of the complex inelastic behaviour of the whole frame system, including the joint panel zone is a complex but essential task [13, 15, 16]. In particular, alternative approaches for modelling the RC beam–column joint, ranging from simplified empirical or macro-models (single- or multi-spring models) to refined finite elements models, are available in literature and under continuous development and validation with experimental results. A detail discussion on the topic is out of the scope of this contribution. Recent proposals including overviews and summary of available methods/models can be found in References [13, 16]. Comparative performance of beam–column joints as well as multi-storey frames for as-built (pre-1970s design) and for retrofitted configurations were carried out through either push–pull (cyclic) or non-linear time-history numerical analyses. Numerical results based on the aforementioned lumped plasticity model [13] confirmed the feasibility and high efficiency of the overall retrofit strategy in protecting the panel zone (as well as the other structural elements) from shear damage or failure, inverting the hierarchy of strength by changing the load path and developing stable plastic hinges in the beams. The overall seismic response of the retrofitted frames was also significantly enhanced.

5. IMPLEMENTATION AND EXPERIMENTAL VALIDATION OF THE RETROFIT SOLUTION

5.1. Experimental program

A series of experimental tests on $\frac{2}{3}$ scaled exterior beam–column joints, in a 2-D configuration, for different details for the haunch systems have been carried out in the Structural Laboratory of

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the University of Canterbury to verify the constructability and efficiency of the proposed solution and to validate the design procedure.

Experimental results from quasi-static tests on three exterior beam–column joints (specimens THR1, THR2, THR3) are herein reported and critically discussed by comparing the observed damage and performance of a benchmark (as-built) specimen, TDP2, representative of general construction practice in the 1950s and 1960s in most of seismic prone counties and tested as part of a more comprehensive research investigation on the seismic behaviour of under-designed beam–column joints with alternative structural details in the panel zone region [14]. Figure 6 and Table I show the specimen geometric properties and reinforcement details. Plain round bars (Grade 300) with end hook anchorage and one single stirrup in the joint panel zone. It is worth recalling that the overall scope of the research investigation was in fact to implement and validate the efficiency of a general retrofit solution for under-designed buildings (regardless of the aforementioned differences in structural detailing from country to country, which typically depends on national code provisions as well as on the local construction and design practice), able to protect the panel zone region from a brittle shear failure mechanism, while targeting the inversion of the hierarchy of strength towards a weak-beam, strong column system. For such reasons, the structural details of the as-built specimen were thus selected to generally represent older construction practice, where a typical brittle behaviour of the joint region would be expected in combination with high beam-to-column

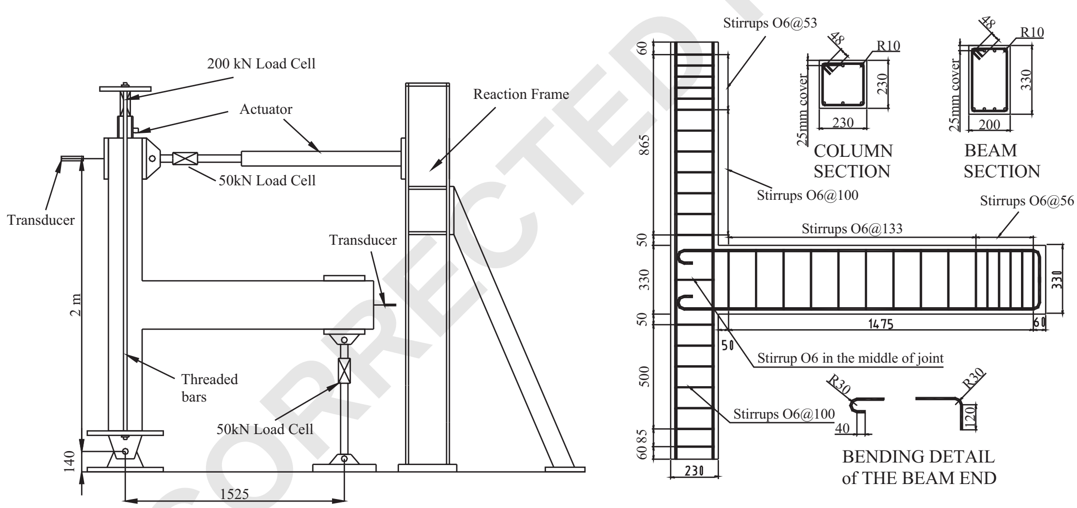

Figure 6. General test set-up and specimen properties.

Table I. Reinforcement properties of specimens.

	Longitudinal reinforcement	Transverse reinforcement
Beam (200 × 330 mm)	4 + 4 R10	R6@133 mm
Column (230 × 230 mm)	3 + 3 R10	R6@100 mm (one in joint)

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Table II. Concrete compression strength.

Specimen	f_c' at 28 days (MPa)	f_c' at day of testing (MPa)
TDP2	21.3	23.3 (50 days)
THR1	24.8	25.9 (56 days)
THR2	24.8	25.9 (83 days)
THR3	22.1	26.8 (56 days)

Table III. Properties of reinforcing steel (Grade 300).

Specimen		Bar size (mm)	f_y (MPa)	ε_y (%)	f_u (MPa)	E-modulus (GPa)
TDP2	Longitudinal	10	333	0.15	467	219
	Transverse	6	408	0.21	482	191
THR1	Longitudinal	10	344	0.15	478	228
	Transverse	6	396	0.2	485	198
THR2	Longitudinal	10	341	0.156	480	218
	Transverse	6	396	0.2	485	198
THR3	Longitudinal	10	347	0.158	474	219
	Transverse	6	352	0.157	436	224

moment capacity ratios to further challenge the inversion of the hierarchy of strength. Further discussion on the effects of alternative structural details in under-designed beam–column joints on the local and global damage mechanism can be found in References [11, 13, 14] (Tables II and III).

5.2. *Test set-up and loading regime*

The quasi-static cyclic tests were carried out under an increasing level of lateral displacement applied at the top of the column. The test set-up and loading protocol are shown in Figures 6 and 7. Beam and column elements were pinned at the assumed inflexion points (assumed to be at mid-span of the beams and at mid-height of the columns). Simple supports at the beam ends were obtained connecting pin-ended steel members to the laboratory floor. Cyclic horizontal lateral loading was applied to the top of the columns using a hydraulic actuator in displacement control. The lateral loading history consisted of a series of two cycles at increasing drift levels followed by a small cycle at a 0.2% drift level. Furthermore, in order to reproduce the effects of a cyclic push–pull test on a frame system, the axial load in the column F_h was varied during the experiments as a function of the lateral load, according to a linear relationship derived from preliminary analyses on the prototype frame system (see Figure 7(b)). The varying axial load on the column represented which affects column flexural capacity as well as joint shear capacity, allowed for a more realistic evaluation of the hierarchy of strength and sequence of mechanisms.

5.3. *Response of the as-built benchmark specimen TDP2*

The response of the as-built or benchmark specimen TDP2 (Figure 8) confirmed the weakness of the beam–column joint panel zone observed in previous tests presented in the literature. First shear

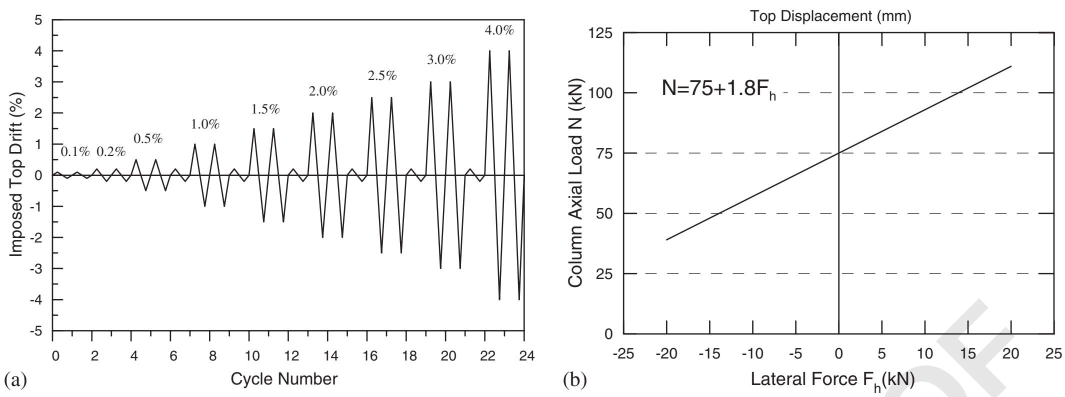

Figure 7. Loading history: (a) imposed lateral drift; and (b) variation of axial load with lateral force.

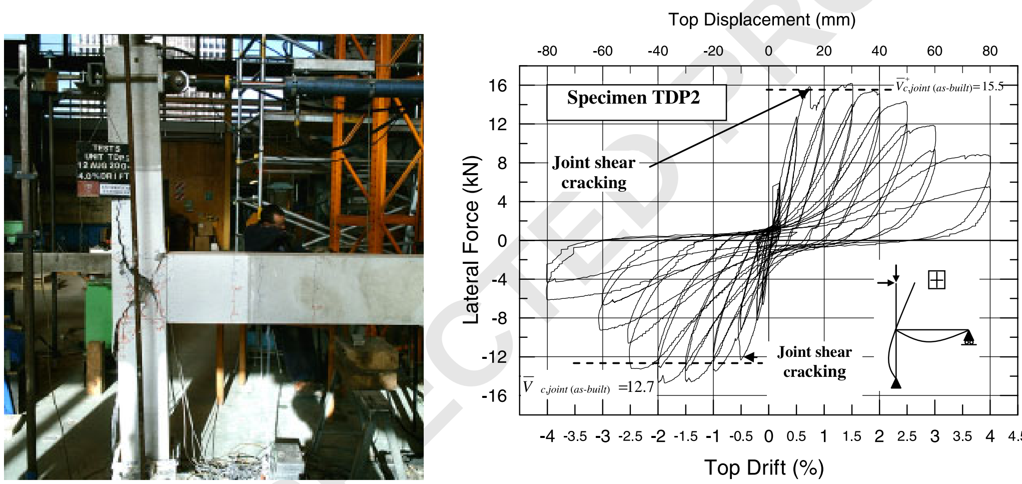

Figure 8. Global damage and hysteresis loop of the as-built specimen.

cracking in the joint region occurred at around 0.5% drift level followed by increased joint damage at increasing level of drift. No flexural damage occurred in the beams and columns. As a result of the formation of a shear hinge mechanism, gradual loss of strength occurred beyond 1.5% drift with a marked pinching of the hysteretic curves (see Figure 8). It is worth noting that the presence of one stirrup in the joint allowed for a more gradual reduction of strength when compared to the sudden and rapid degradation observed in previous tests with no transverse reinforcement at all in the joint [6].

5.4. Design of the haunch elements

The design of the haunch retrofit solution system for all specimens was carried out following the conceptual procedure outlined in the previous paragraphs, with the intention of protecting

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the joint region from excessive damage while forcing a plastic hinge in the beam away from the column interface (at the outer side of the haunch connection). Beam and column members had to also be protected against excessive shear demand and brittle failure by controlling the haunch design parameters. Due to several uncertainties that were expected in the actual implementation of the haunch device, the experimental testing and the specimen properties, a conservative design of the haunch device was followed in order to guarantee the desired hierarchy of strength while maintaining proper 'margins' between critical events. As a result, assuming $L' = 400$ mm and $\alpha = 45°$, the targeted design value for the haunch stiffness was $K_d = 100\,000$ kN/m, corresponding to 'actual' safety factors Φ_1 and Φ_2 of about 0.7 and 0.85, respectively (see Table V).

The haunch system consisted of elastic elements for retrofitted specimens THR1 and THR3 and of a yielding fuse element for retrofitted specimen THR2. The haunches were connected to the concrete elements through either a hinged base for THR1 and THR2 or a welded base for THR3. Two external rods, partially prestressed to guarantee proper anchorage of the whole haunch solution to the structural elements, were used, in addition to two anchors directly fastened to both the beam and to the column (see Figure 9). The haunch axial element was obtained by machining down deformed bars for a designed length and then inserting them into steel grouted tubes adopted as anti-buckling systems. The mechanical properties of the different haunch elements are summarized in Table IV. It is worth noting that the expected equivalent stiffness ($K_d \cong 100\,000$ kN/m)

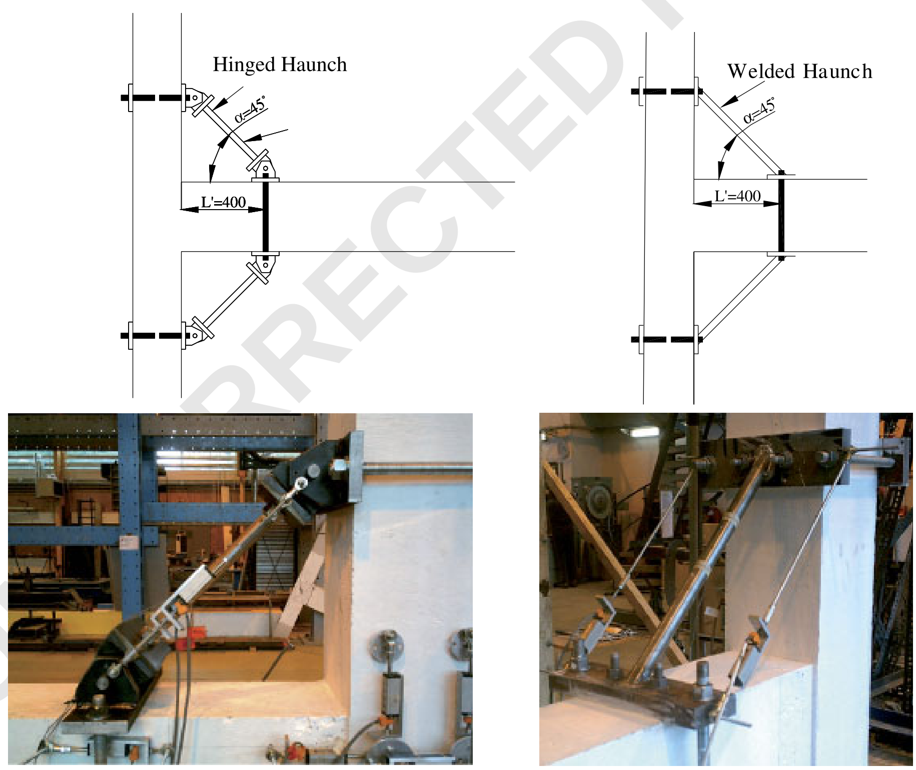

Figure 9. Alternative connection details for the haunch system: hinged (left side, implemented in specimens THR1 & THR2) or welded (right side, implemented in specimen THR3).

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Table IV. Properties of haunch elements.

Specimen	Steel grade	E-modulus (MPa)	f_y (MPa)	Fuse diameter (mm)	F_y (kN)	Equivalent axial stiffness K_d (kN/m)	Connection type
THR1	430	176	498	14	76	107 000	Hinged
THR2	300	195	340	9	22	109 000	Hinged
THR3	300	235	325	19	90	110 000	Welded

Table V. Expected hierarchy of strength and actual 'margin' or safety factors Φ_i between sequence of events (design parameters: $L' = 400$ mm, $\alpha = 45$, $K_d = 100\,000$ kN/m, Φ_1 and $\Phi_2 = 0.85$).

Limit on $\frac{p_t}{\sqrt{f'_c}}$	K_d (kN/m)	β	$\overline{V}_{\text{c,beam-hinge}}$ (kN)	$\overline{V}_{\text{c,col-hinge}}$ (kN)	$\overline{V}_{\text{c,joint}}$ (kN)	$\frac{\overline{V}_{\text{c,beam-hinge}}}{\overline{V}_{\text{c,col-hinge}}}$ $(\leqslant \Phi_1)$	$\frac{\overline{V}_{\text{c,col-hinge}}}{\overline{V}_{\text{c,joint}}}$ $(\leqslant \Phi_2)$	$\frac{\overline{V}_{\text{c,beam-hinge}}}{\overline{V}_{\text{c,joint}}}$ $(\leqslant \Phi_1\Phi_2)$
0.19	100 000	2.1	25	35	41.5	0.71	0.84	0.6
0.29	100 000	2.1	25	35	50.7	0.71	0.69	0.49

refers to the haunch axial element itself without including the effects of the hinged or welded base connections. These effects were included in the design process by using appropriate safety factors Φ_i.

The yielding haunch was designed for a yield force $F_y = 20$ kN. This value of F_y was chosen to be slightly lower than the expected maximum axial force in the elastic haunch system. This force was also measured experimentally in the experiment THR1 as $F_{\max} = 25$ kN. As discussed by Christopoulos and Filiatrault [19], in the case of a yielding haunch, the system behaves similarly to systems with non-yielding haunches (described in the previous paragraphs) until the forces in the passive haunches reach the strength of the device F_y. Up to that point, the moment and shear in the beam follow the diagram shown in the top row of Figure 4 and are dependent on the elastic properties and geometry of the haunches. When the haunches reach their yield load, assuming they do not exhibit significant post-yielding stiffness, any additional lateral loads applied to the system will cause internal forces following the distribution presented in Figure 4 (without haunch). After the device reaches its yield load F_y, the moment in the beam at the face of the column will increase at a much higher rate than it does before the haunch yields. If the beam does not form a plastic hinge at the location where the haunches are attached after the devices have slipped or yielded, the joint will suffer damage. Considering this, F_y must be chosen such that when the devices yield, the moment at the location where the haunches are attached to the beam is sufficiently close to the plastic moment of the beam to assure that the damage to the joint does not occur before yielding of the beam. In general terms, for the elasto-plastic haunches, the design is first carried out assuming elastic elements following the procedure described above and then the lowest value of F_y that assures yielding of the beam before the joint is damaged is determined.

5.5. Experimental results of the retrofitted solutions

5.5.1. Response of THR1 (elastic haunch with hinge connection). Figure 10 shows the behaviour of the specimen retrofitted with the elastic haunch system (THR1). As targeted in the design

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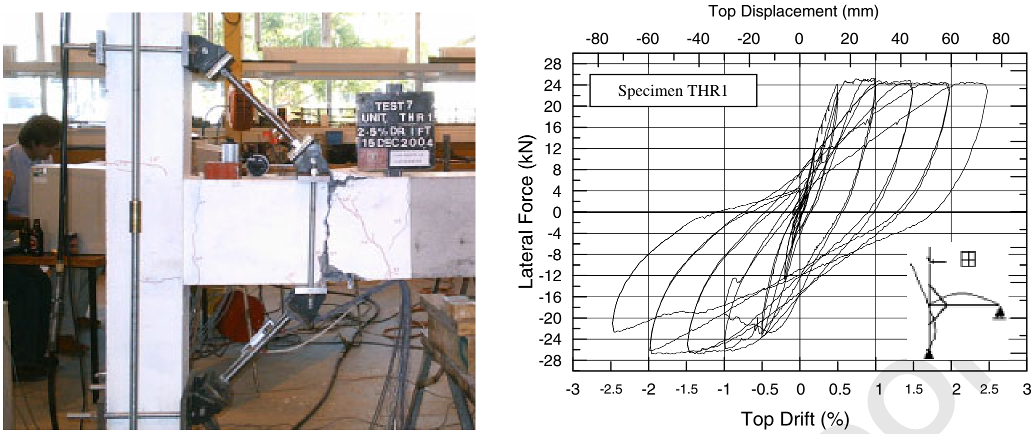

Figure 10. Experimental behaviour of specimen THR1 (elastic hinged haunch): relocation of plastic hinge and hysteresis loop.

process, no shear damage occurred in the joint panel zone region, while a more desirable flexural hinging behaviour within a weak-beam strong column mechanism was observed. The flexural hinge was established by the formation and progressive widening of a main flexural crack at the haunch-to-beam connection (this localized hinging mechanism is typical of beam section with plain reinforcement). As a result, a more stable hysteretic response with increased energy dissipation was observed when compared to the as-built specimen response (see Figure 8). Moreover, due to the relocation of the plastic hinge away from the column interface, the longitudinal beam bars can rely on a full anchorage/development length as shown by the limited pinching effect in the hysteresis.

When comparing the improved global hysteretic behaviour of the retrofitted configuration to the as-built solution (see Figure 8), it is also worth noting that, as a result of the inversion of the hierarchy of strength, a substantial increase in the global subassembly lateral strength is observed. This increase of strength is not due to the increase of strength of any individual member of the system but rather to the inversion of the hierarchy of strength of the same members. More specifically, the lateral load capacity of the as-built specimen, corresponding to the occurrence of a shear hinge mechanism ($\overline{V}_{\text{c,joint(as-built)}}$, calculated according to the internal force distribution for the as-built configuration) has increased, thanks to the retrofit solution, to the value of lateral load capacity corresponding to the formation of a plastic hinge in the beam ($V_{\text{c,beam-hinge}}$ as per Equation (10)).

5.5.2. Response of THR2 (yielding haunch with hinge connection). The experimental response of the THR2 specimen, which incorporated a yielding haunch solution with a hinge connection, also achieved the targeted protection of the panel zone as well as the reversal of the strength hierarchy to force a flexural hinge in the beam. As in the previous test (THR1), a stable hysteretic response with good energy dissipation was observed with marked pinching occurring only at higher drift levels (beyond 2%) due to the wide opening/closing of the main flexural crack a the beam/haunch connection interface which caused some shear sliding. Based on the global hysteretic response and the strain gauge readings on the yielding haunch element, the dissipating solution did not seem

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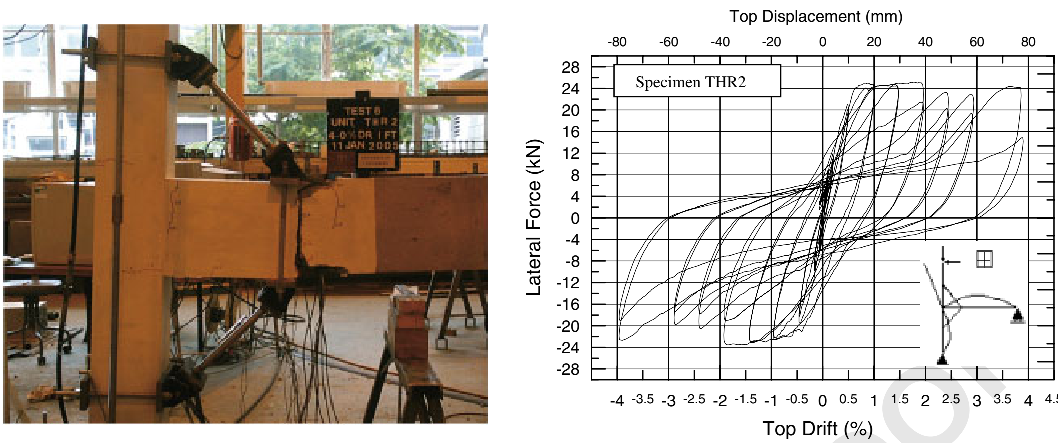

Figure 11. Experimental behaviour of specimen THR2 (yielding hinged haunch): relocation of plastic hinge and hysteresis loop.

to be particularly effective in improving the energy dissipation of the system, when compared to the elastic solution adopted in the THR1 specimen. Due to the characteristics of these pre-1970s RC frame systems, a complex trade-off between a high haunch stiffness to activate the haunch before excessive rotation demand occurs in the joint and the need to provide a sufficient displacement excursion of the haunch device to guarantee additional hysteretic energy dissipation (while still protecting the joint region from excessive damage) is difficult to achieve. For the system in this experimental study, the use of a dissipating haunch was therefore not a viable solution (Figure 11).

5.5.3. Haunch stiffness in THR1 and THR2 specimens. The first version of the haunch system, implemented in the specimens THR1 and THR2, was based on the use of a hinged connection (see Figure 9, left). Due to the flexibility of the hinged base supports and to the tolerances within the mechanical connections, the actual global stiffness of the haunch device (i.e. haunch plus base hinges and plate), measured experimentally, was significantly lower than the 'factored' design value ($K_{d\,\mathrm{experimental}} \cong 22\,000$–$25\,000\,\mathrm{kN/m}$ instead of the targeted $K_d = 100\,000\,\mathrm{kN/m}$). However, thanks to the conservatism built into the design approach through higher safety factors accounting for different uncertainties expected at a first stage, both the THR1 and THR2 solutions achieved the major goal of inverting the hierarchy of strength while protecting the panel zone region.

5.5.4. Response of THR3 (elastic haunch with welded connection). In the second phase of the experimental program, focus was set on improving the constructability and effectiveness of the haunch. This led to the development of a second generation solution for the haunch elements. By welding the haunch axial element directly to the plate connection (see Figure 9, right), stiffness losses in the haunch connections were eliminated. The equivalent stiffness of the slightly longer haunch element was kept equal to the targeted $K_d = 100\,000\,\mathrm{kN/m}$ to allow for comparisons with specimens THR1 and THR2.

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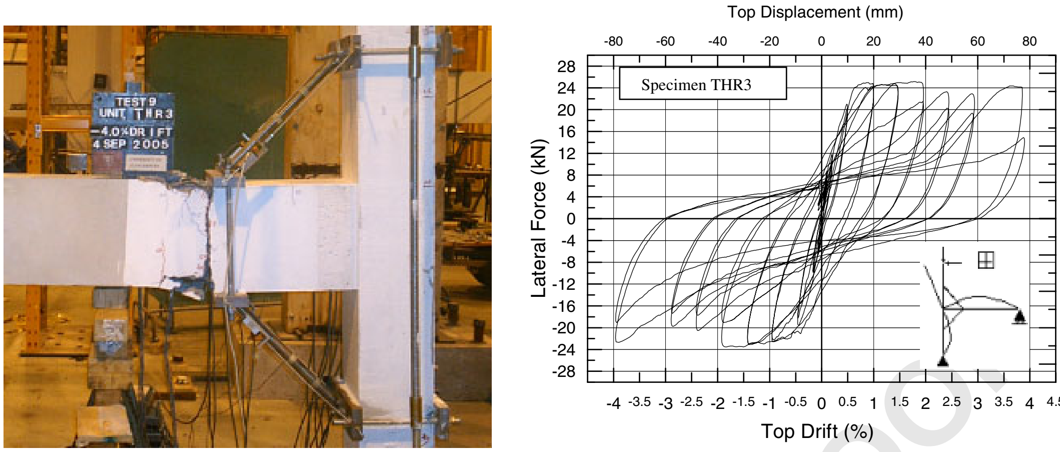

Figure 12. Experimental behaviour of specimen THR3 (elastic welded haunch): relocation of plastic hinge and hysteresis loop.

As shown in Figure 12, the THR3 retrofit intervention with the improved haunch detail was successful in forcing a plastic hinge to form in the beam and in protecting the panel zone region. However, unlike tests THR1 and THR2, the global haunch stiffness measured experimentally was almost the same as the targeted design value of $K_d = 100\,000$ kN/m. This second generation haunch solution allows for a very reliable control of the haunch stiffness, and therefore the calculations of internal forces used for the design of the retrofit solution can be done more adequately. Reducing the conservatism that was built in to the design of THR1 and THR2 specimens (by increasing the safety factors) increases the number of feasible haunch configurations that can be considered.

5.5.5. Redistribution of deformation demand and fuse effect of the beam hinge. A further confirmation of the efficiency of the proposed retrofit solution in protecting the panel zone region from excessive damage while developing a plastic hinge in the beam, is given in Figure 13, where the experimental relative contributions to the total drift from beam–column and panel zone region are shown, for the as-built and the retrofitted specimens.

While in the as-built solution (TDP2 specimen) the occurrence of a shear hinge mechanism leads to a concentration of the subassembly rotation demand in the panel zone, while column and beam deformation/rotation demand remain limited, the opposite behaviour is observed in the retrofitted configurations. The development of a flexural mechanism (plastic hinge) in the beam acts as a protective fuse for the panel zone. A quantitative summary of the experimental results in terms of subassembly overall strength, initial stiffness and energy dissipation capacity of the as-built and retrofitted configurations is given in Table VI as further confirmation of the efficiency of the proposed retrofit solution. It can be noted that the haunch solution leads to an appreciable increase in the lateral strength of the subassembly from values in the order of 14–16 kN, corresponding to shear damage and failure of the joint panel zone region, to values of 23–26 kN (increase of 55 and 83%), corresponding to the development of a plastic hinge in the beam in line with the desired and more favourable weak-beam strong column inelastic mechanism of the overall frame system.

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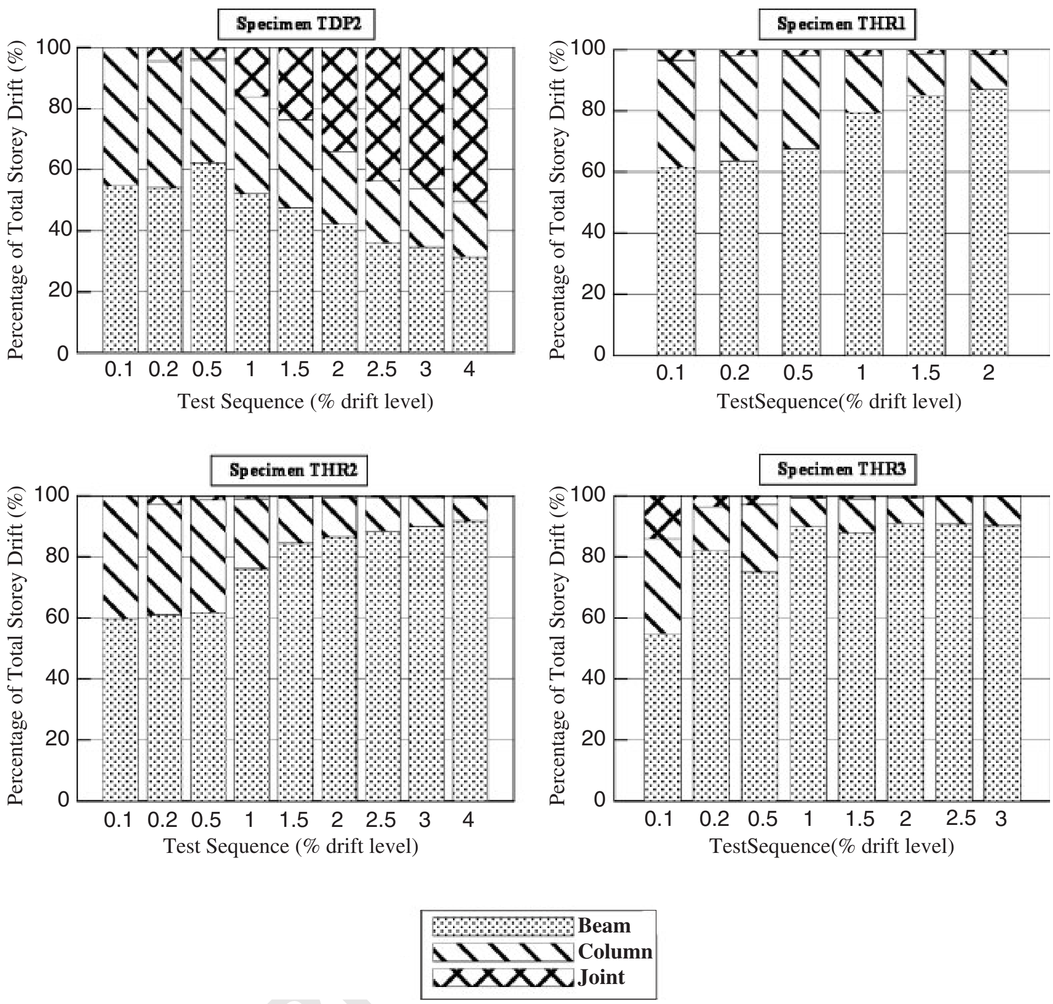

Figure 13. Contributions to subassembly drift of beam, column and joint in as-built and retrofitted solutions (positive loading direction).

The more stable flexural mechanism in the beam also allows for a substantial improvement in the energy dissipation capacity of the system, thus further reduction of the displacement demand, with equivalent viscous damping values, ξ, moving from 13–15% (at 1 and 2% of drift) to 21–25% (increase of 45–93%). The consistently observed increase in the 'initial' stiffness of the subassembly (evaluated as elastic stiffness of an equivalent bilinear force–displacement envelope) from approximately 1.5–1.6 to 2.7–3.1 kN/m (+70–100%), could also provide beneficial effects in limiting excessive deformations even under moderate earthquake ground motions. Preliminary numerical results on frame systems retrofitted with haunch solutions [24] have also shown that there is only a marginal increase in the floor accelerations as a result of the installation of stiffening haunches.

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Table VI. Summary of experimental response of as-built and retrofitted configurations.

Specimen	Initial stiffness (kN/mm)		Ultimate strength (kN)		Equivalent viscous damping (%)			
	Positive loading	Negative loading	Positive loading	Negative loading	0.5% drift	1% drift	2% drift	3% drift
TDP2 (as-built)	1.60	1.56	15.95	14.3	15.0	14.5	13.5	15.0
THR1	3.06	3.00	24.5	26.1	10.9	23.5	26.2	—
THR2	2.70	2.75	25.2	23.2	11.7	21.2	25.6	18.7
THR3	3.12	3.10	25.0	25.2	13.3	21.0	26.3	20.3

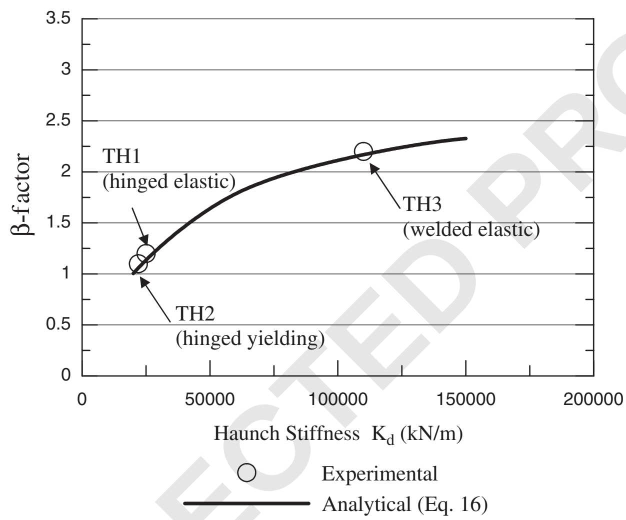

Figure 14. Analytical–experimental comparison of β-factor (haunch contribution).

5.6. Validation of the design procedure with experimental results

The efficiency of the overall retrofit strategy in terms of controlling the hierarchy of strength and sequence of mechanisms can be further appreciated by comparing the experimental results with the expected distribution of internal forces in the beam and column, the joint shear demand (in terms of either nominal shear force V_{jh}, principle tensile stress p_{t}, or equivalent moment $M_j = M_{\mathrm{bc}}$) as well as with predicted lateral load capacities corresponding to the occurrence of each mechanism. To assess the validity of the proposed analytical approach, actual material properties, along with the actual values of the haunch global stiffness (as measured experimentally) were used to back-calculate the responses of the retrofited specimens which were then compared to the experimentally measured responses. The formulation given in Equation (16) for the calculation of the β-factor was used to account for column flexibility. Figure 14 shows the comparison between the β-factors derived from experimental measurements with the values calculated using Equation (16) with the properties of the tested haunches.

A more complete comparison between analytically derived and experimental results is presented in Table VII, in terms of (i) the β-factor, (ii) the level of principle tensile stresses, p_t, developed in the joint (normalized by $\sqrt{f'_c}$), (iii) the beam moments at the joint interface, M_{bc}, (iv) the beam moments at the haunch connection, $M_{b(max)}$, and (v) the subassembly lateral force capacity corresponding to the development of the desired plastic hinge event, $\overline{V}_{c,\text{beam-hinge}}$. As can be seen in this table, the proposed analytical procedure captures well all the measured responses. Provided that appropriate mechanical connection details are adopted for the haunch device to limit the observed reduction of stiffness as well as the slackness of the connection (i.e. welded solution versus hinged), the effects of the overall retrofit solution can be controlled in the design with good confidence.

To study the effect of variable transverse reinforcement spacing, to represent lower and upper bounds in older construction practices a comparison between shear demand and capacity of columns and beam elements in the (scaled specimen) retrofitted specimens was carried out according to the NZS3101:2005 guidelines [21] assuming an increase of 50 and 100% in the transverse reinforcement spacing and is presented in Table VIII. It can be noted that with the 100% increase in spacing, shear capacities are still sufficient to resist the increased levels of shear forces. However, as indicated in the step-by-step design procedure presented in the previous paragraphs, a detailed analysis must carried to assure that the desired hierarchy of strength is satisfied, with acceptable margins of safety for undesirable mechanisms such as shear failures.

Table VII. Analytical–experimental comparison of the efficiency of the haunch retrofit solution.

Specimen	K_d (kN/m)		β	$\frac{p_t}{\sqrt{f'_c}}$	M_{bc} (kNm)	$M_{b(max)}$ (kNm)	$\overline{V}_{c,\text{beam-hinge}}$ (kN)
THR1	25 000	Analytical	1.15	0.22	22.4	30	22.8
		Experimental	1.2	0.21	24.3	31.7	23.7
THR2	22 000	Analytical	1.07	0.24	23.9	30	22.8
		Experimental	1.1	0.19	20.8	29	22.14
THR3	110 000	Analytical	2.17	0.06	4.9	30	22.8
		Experimental	2	0.10	10.3	31.1	24.3

Table VIII. Verification of shear capacity versus demand of columns and beam elements in the retrofitted configuration assuming variable transverse reinforcement (spacing).

				Shear capacity (kN)			
Specimens		Shear demand (kN)	Transverse reinforcement spacing s (mm)	V_s	V_c	Total (PPHZ)	Total (outside PPHZ)
THR1,THR2, THR3	Column	24	$s=100$	46	30	46	76
			$s=150$	31		31	61
			$s=200$	23		23	53
	Beam	32	$s=133$	53	37	53	90
			$s=200$	35		35	72
			$s=266$	26		26	63

Note: Evaluation of shear capacity according to NZS3101 [21]. PPHZ refers to a potential plastic hinge zone where the shear contribution from concrete, V_c, is neglected.

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6. CONCLUSIONS

A simple and effective retrofit strategy for retrofitting existing reinforced concrete buildings designed mostly for gravity loads prior to the introduction of modern seismic code provisions has been presented. A simplified design approach based on an analytical formulation for the forces in the beam–column system retrofitted with the proposed haunch system has been suggested. The goal of the proposed solution is to reverse the hierarchy of strength by forcing a plastic hinge in the beam while protecting the panel zone from excessive cracking and strength degradation. An experimental program was carried out to verify the effectiveness of the proposed technique to significantly enhance the response of the beam–column subassemblies, but also to validate the proposed analytical formulation and design procedure.

Results from a control benchmark test were first presented where the deficiencies related with the joint panel zone shear damage were confirmed. The design and practical implementation of a simple haunch element consisting of a threaded steel bar fastened to hinged plates that are connected to the beams and columns was then carried out. The retrofitted specimens displayed a substantially enhanced response when compared to the non-retrofitted specimens: damage to the joint was eliminated and a flexural plastic hinge formed in the beam at the location of the beam–haunch connection. This resulted in an increase in the system lateral strength, a stable hysteretic behaviour and enhanced energy dissipation capacity. The second of these specimens was designed to yield at a force slightly lower than the maximum force developed in the first specimen to enhance the energy dissipation capacity of the system. For this tested configuration, the yielding of the haunch element did not significantly enhance the performance of the proposed retrofit strategy although it did meet the design requirement of protecting the panel zone while forming a flexural hinge in the beam. To further improve the performance of the haunch, a second generation detail which used a welded haunch was then built and validated through a third experiment. The third experiment also achieved the desired response of beam flexural hinging and protection of the panel zone. The stiffness of the haunch element measured in this third experiment was very close to the expected value. Results obtained using the proposed analytical derivations were very close to results measured experimentally thus validating the steps that are suggested for the design of the proposed retrofit solution.

Further studies on the application of this retrofit technique to other types of non-seismically designed RC frames (i.e. shallow and wide beam, flat slabs) and on the practical definition of alternative elastic or dissipating haunch elements are needed. Experimental investigations on the local behaviour of the haunch system including appropriate fastening solutions to the existing frame are currently underway. Finally, investigations on the multiple aspects of the global response (3-D bi-axial, larger assemblies with floor systems) of systems retrofitted with the proposed technique are also being carried out.

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