Retrofitting RC Beam-Column Joint in Australia using Single Diagonal Haunch

Alireza Zabihi[1,5], Hing-Ho Tsang[2,5], Emad F. Gad[3,5] and John L. Wilson[4,5]

1. PhD Candidate, Department of Civil and Construction Engineering, Swinburne University of Technology, Melbourne, Australia.
   Email: azabihi@swin.edu.au

2. Corresponding Author. Senior Lecturer, Department of Civil and Construction Engineering, Swinburne University of Technology, Melbourne, Australia.
   Email: htsang@swin.edu.au

3. Professor and Chair, Department of Civil and Construction Engineering, Swinburne University of Technology, Melbourne, Australia.
   Email: egad@swin.edu.au

4. Professor and Executive Dean, Faculty of Science, Engineering & Technology, Swinburne University of Technology, Melbourne, Australia.
   Email: jwilson@swin.edu.au

5. Bushfire & Natural Hazards Cooperative Research Centre, Melbourne, Australia.

Abstract

Exterior beam-column joint is potentially the weakest link in a limited-ductile RC frame structure in Australia. The use of diagonal haunch element has been considered as a desirable seismic retrofit option for preventing brittle failure of the joint. Previous research globally has focused on implementing double haunches, whilst the performance of using single haunch element as a less-invasive and more architecturally favourable retrofit option has not been investigated. In this study, the feasibility of using a single haunch system for retrofitting RC beam-column joint in Australia is explored. This paper presents the key formulations of the technique and illustrates its effectiveness by showing analytically the changes in the shear demand at the joint.

Keywords: limited-ductile, RC frame, exterior beam-column joint, seismic retrofitting, single haunch.

1. INTRODUCTION

A large number of habitable non-seismically designed RC frame buildings exist all over the world including Australia. Undesirable brittle failure is expected to occur in this kind of buildings in an event of major earthquake. Previous analytical and experimental studies proved that limited-ductile beam-column joint is the most vulnerable structural element subjected to lateral loads (Aycardi et al., 1994; Beres et al., 1996; Calvi et al., 2002). This deficiency generally arises from poor detailing in the joint area and consequently lack of capacity design principles in the overall response of the structures

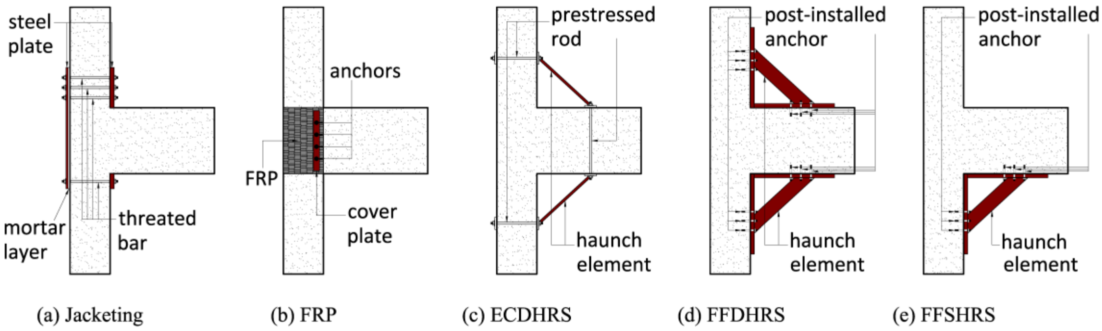

Figure 1: Schematic diagrams of various retrofit techniques for external RC beam-column joint: (a) Steel jacketing; (b) Fibre-reinforced polymer; (c) Externally clamped double haunch retrofitting system; (d) Fully fastened double haunch retrofitting system; (e) Fully fastened single haunch retrofitting system.

(Pampanin et al., 2006). To improve the global seismic behaviour of the structure, enhancement of the weakest links is essential which can be achieved by seismic retrofitting. In recent years, various retrofit techniques such as strengthening of joint (e.g. steel jacketing (Figure 1(a)), fibre-reinforced polymer (FRP) (Figure 1(b))) and relocating the plastic hinge away from the joint (e.g. externally clamped double haunch retrofitting system (ECDHRS) (Figure 1(c)), fully fastened double haunch retrofitting system (FFDHRS) (Figure 1(d))) have been investigated and utilised as practical solutions (Beres et al., 1992; Ghobarah and Said, 2002; Chen, 2006; Genesio, 2012).

Strengthening by steel jacketing or FRP and the ECDHRS could be conveniently installed in laboratory tests, but these are challenging to be implemented in practice because of limited accessibility to the joint zone due to the presence of wall and floor slab. Although this limitation has been eliminated by using post-installed mechanical anchors in the FFDHRS (Sharma et al., 2014), the use of upper diagonal haunch (located on the floor) still remains as an aesthetic and functional restriction. Hence, the fully fastened single haunch retrofitting system (FFSHRS) (Figure 1(e)) is proposed herein this paper as a preferred alternative.

2. CASE STUDY BUILDING

A full-scale three-storey RC moment resisting frame considered in this study has been designed based on the requirements in the 1980's (as shown in Figure 2(a)). The frame is 9 m tall, 10 m wide, and is located on a deep or very soft soil site (i.e. Class D or E as defined in AS1170.4-2007) in Melbourne. The seismic weight was calculated by assuming 10 kPa gravity loads for all three levels including dead loads and 30% of imposed loads. A two-dimensional single frame model, with half of the bay on each side (4 m in total), is considered.

Seismic Base Shear

The design equivalent static shear force, V_{Base}, at the base of this frame model is calculated from the following equation in accordance with AS1170.4-2007:

$$V_{Base} = \left[k_p Z C_h(T_1)\frac{S_p}{\mu}\right] w_t \qquad 1$$

where k_p = probability factor (= 1.0); Z = earthquake hazard factor (= 0.08); $C_h(T_1)$ = spectral shape factor for the fundamental natural period of the structure (i.e. 3.68 for T_1

= 0.49s); μ = structural ductility factor (= 2.0); S_p = structural performance factor (= 0.77); and w_t = total design seismic weight of the building (= 1200 kN).

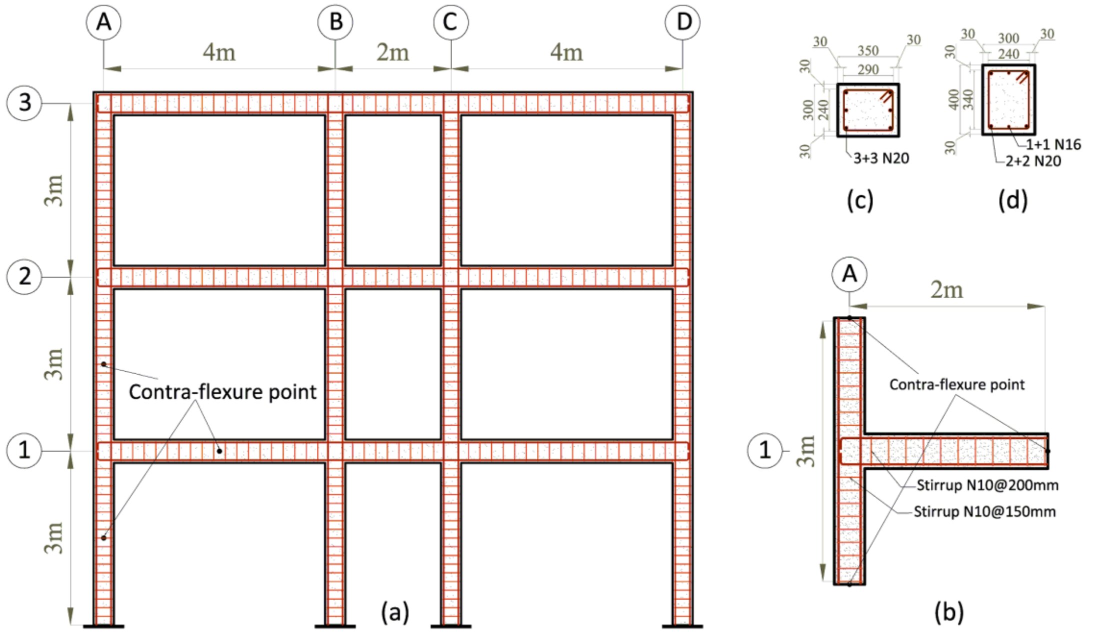

Figure 2: Geometry of case study model: (a) Full-scale RC moment resisting frame; (b) Exterior Beam-Column Joint; (c) Column section; (d) Beam section.

The base shear forces based on the design ultimate limit state (ULS) (with the consideration of over-strength and ductility), 500-year and 2500-year return period elastic response (ER) are approximately equal to 135 kN, 350 kN, and 630 kN respectively. Hence, the shear force at the base of the first storey exterior column would be 27 kN (design ULS), 70 kN (500-year ER) and 126 kN (2500-year ER) respectively. The natural periods of the structure will be slightly decreased due to the stiffening effects by the haunches. It may lead to an increase in the base shear force, but it is not realised in this study as the initial fundamental natural period is within the peak acceleration plateau of the design response spectrum in AS1170.4-2007.

Exterior Beam-Column Joint Subassembly

To investigate the effectiveness of the proposed retrofitting system, the bottom left beam-column joint subassembly, annotated as A1 in Figure 2, has been selected to be assessed analytically (shown in Figure 2(b)). This subassembly is truncated between contra-flexure points at mid-height of the columns and mid-span of the beam. Cross sections of column and beam are shown in Figure 2(c) and Figure 2(d), respectively. The key parameters for analytical modelling are tabulated in Table 1 (material properties) and Table 2 (geometry of the joint subassembly).

Table 1: Material Properties

Concrete	f'_c = 25 MPa	E_c = 26700 MPa	α_2 = 0.85	γ = 0.85	ε_{cu} = 0.003
Reinforcement	f_y = 500 MPa	E_s = 200 GPa			

Noted that f'_c = the characteristic compressive strength of concrete at 28 days; E_c = the modulus of elasticity of concrete at 28 days; α_2 = the ratio of equivalent concrete compressive stress developed under flexure to the characteristic compressive strength (f'_c); γ = the ratio, under design bending or design combined bending and compression, of the depth of the assumed rectangular compressive stress block to $k_u d$; whereas d = effective depth of a cross-section, and k_u = neutral axis parameter being the ratio, at ultimate strength under any combination of bending and compression, of the depth to the neutral axis from the extreme compressive fibre to d; ε_{cu} = the ultimate concrete strain; f_y = yield strength of steel reinforcing bar; and E_s = the modulus of elasticity of steel.

Table 2: Geometry of the exterior beam-column joint (A1) subassembly

Beam		**Column**		**Joint**	
h_b =	400 mm	h_c =	350 mm	w_j =	300 mm
w_b =	300 mm	w_c =	300 mm	h_j =	350 mm
$L_e = L_b/2$ =	2000 mm	H_e =	3000 mm	Shear Rein.	N/A
Long. Rein.	2 N20 + 1 N16	Long. Rein.	3 N20		
Shear Rein.	N10 / 200 mm	Shear Rein.	N10 / 150 mm		
Cover =	30 mm	Cover =	30 mm		

where h_b = beam section depth; w_b = beam section width; L_e = beam half length; L_b = beam span length between column centrelines; h_c = column section depth; w_c = column section width; H_e = effective height of column between two vertical consecutive inflection points; h_j = joint horizontal section depth; and w_j = joint horizontal section width.

The lateral load – drift capacity relationship of the first storey exterior column has been calculated based on the analytical model proposed recently by Wibowo et al. (2014) and Wilson et al. (2015). The peak shear strength, under an axial load ratio of 0.1, is 110 kN, which is higher than the 500-year ER (70 kN) levels. This indicates that the column will respond within the pre-peak range with drift demand less than 1.0%, for the range of seismic actions being considered in this study, whilst the ultimate drift capacity of the column is over 5.0%. It will be shown in Section 6 that tensile crack will likely form at the joint (when the tensile stress is being exceeded, which is assumed as un-repairable damage and is defined as joint failure in this study, although it does not necessarily lead to collapse) or plastic hinge will form at the beam before the column reaches its peak shear capacity.

3. SHEAR DEMAND AT EXTERIOR BEAM-COLUMN JOINT

Based on the applied shear and axial forces, V_c and N_c (as shown in Figure 3), the shear force diagrams of the beam-column joint subassembly for the non-retrofitted system (NRS) and the single haunch retrofitted system (SHRS) are plotted in Figure 4. For each case, the horizontal shear force at the mid-depth of the joint panel zone, V_{jh}, can be expressed as a function of the applied shear force:

$$V_{jh-NRS} = \left[\frac{H_e}{j_b L_e}\left(\frac{L_n}{2}\right) - 1\right] V_c \tag{2}$$

$$V_{jh-SHRS} = \left[\frac{H_e}{j_b L_e}\left(\frac{L_n}{2} - \left(\frac{2b + h_b - j_b}{2\tan\alpha}\right)\beta_{SHRS}\right) - 1\right] V_c \tag{3}$$

where j_b = internal lever arm of the beam section between tension and compression side; L_n = net beam span length between column faces; b = vertical length of the haunch; α = angle between the haunch and the beam; β_{SHRS} = shear transferring factor at the beam for SHRS (refer next section), and all other parameters have been specified previously.

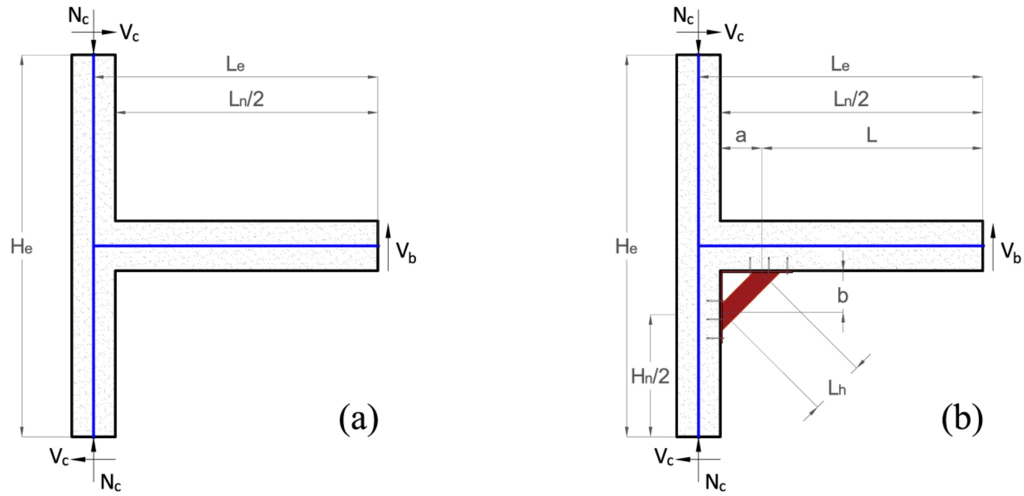

Figure 3: External actions on exterior beam-column joint: (a) Non-Retrofitted System (NRS); and (b) Single Haunch Retrofitting System (SHRS).

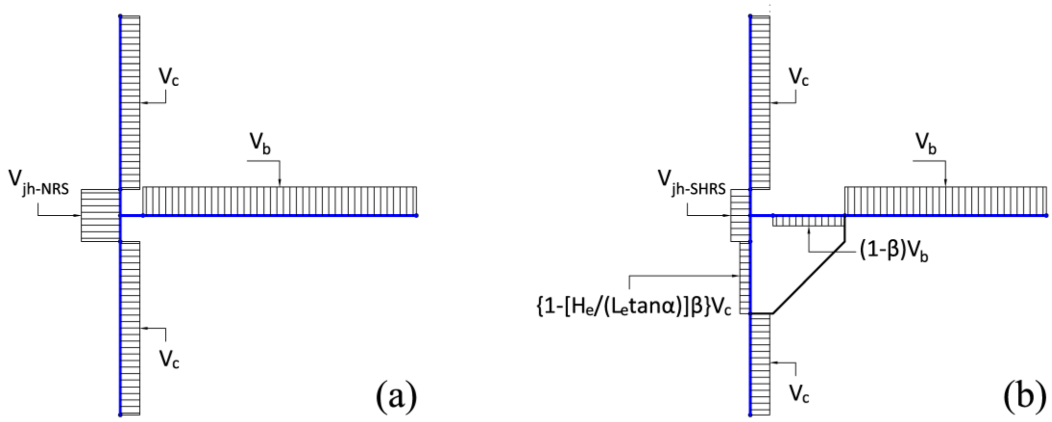

Figure 4: Shear force diagrams: (a) Non-Retrofitted System (NRS); and (b) Single Haunch Retrofitting System (SHRS).

4. SHEAR TRANSFERRING FACTOR

The value of the shear transferring factor, β, can be derived based on the deformation compatibility theory at the haunch-beam and haunch-column connection points (Yu et al., 2000; Pampanin et al., 2006). Zabihi et al. (2016) derived the formulation of β-factor for SHRS by considering both beam and column deformations:

$$\beta_{SHRS} = \frac{N_1 + N_2}{D_1 + D_2 + D_3 + D_4} \tan \alpha \qquad 4$$

where N_1, N_2, D_1, D_2, D_3 and D_4 can be defined as follows:

$$N_1 = 4ab + 3h_b a + 6Lb + 6h_b L$$

$$N_2 = \lambda_1 \lambda_2 (4ab + 3h_c b + 6Ha + 6h_c H)$$

$$D_1 = 4ab \tan \alpha + 3h_b a \tan \alpha + 3h_b b + 3{h_b}^2$$

$$D_2 = 12E_c I_b / (K_h . a \cos^2 \alpha)$$

$$D_3 = \lambda_1 \left(4ab \tan \alpha + 6h_c b \tan \alpha + 3{h_c}^2 \tan^2 \alpha + 12 I_c \tan^2 \alpha / A_c\right)$$

$$D_4 = 12 I_b / A_b$$

$\lambda_1 = I_b b/(I_c a)$

$\lambda_2 = L_e b/(H_e a)$

in which, a = horizontal length of the haunch; I_b = second moment of area of the beam cross section; I_c = second moment of area of the column cross section; K_h = haunch axial stiffness; A_b = area of the beam cross section; A_c = area of the column cross section, and all other parameters have been explained earlier.

5. PRINCIPAL TENSILE STRESS IN EXTERIOR BEAM-COLUMN JOINT

The shear capacity of RC beam-column joint is mainly governed by its principal tensile stress, p_t (Priestley, 1997; Pampanin and Christopoulos, 2003), which is a function of the joint shear stress, v_{jh}, and the compressive stress due to column axial load, f_a. According to Mohr's circle theory, the principal tensile stress, p_t, at mid-depth of the joint core can be calculated by:

$$p_t = -(f_a/2) + \sqrt{(f_a/2)^2 + v_{jh}^2} \qquad 5$$

where $f_a = N_c/(w_c h_c)$ and $v_{jh} = V_{jh}/(w_j h_j)$.

For RC beam-column joint with no transverse reinforcement, the diagonal tension is mainly resisted by concrete. Initial cracking at the joint is estimated to occur when $p_t = 0.29\sqrt{f'_c}$. However, the longitudinal flexural reinforcement in the beam extends into the joint (refer Figure 2(b)), which leads to confinement of the concrete diagonal strut in the joint core and hence, joint shear strength can be enhanced (Sharma et al., 2011). The permissible tensile strength is assumed to be $0.42\sqrt{f'_c}$ (Priestley, 1997), beyond which shear hinge is assumed to have formed at the joint.

6. RESULTS AND DISCUSSIONS

Amongst the possible un-repairable damage types or failure modes which may occur at RC beam-column joint during an earthquake, joint shear failure and beam flexural yielding were found to be the first two limits at this subassembly (A1). Therefore, the joint shear and beam flexural yielding capacity-demand ratios are plotted in Figure 5 against the base shear force, V_{Base}, and its corresponding column shear force, V_c, whereas the column axial force, N_c, is assumed to be constant. Failure is assumed to occur when the capacity-demand ratio is smaller than 1.0, and hence, the "failure" threshold is defined at the point when the ratio equals 1.0. It is noted that three inclinations (i.e. α = 45°, 23°, and 20°) of the haunch were considered, whilst the lengths of haunches were made the same (i.e. same amount of materials) for a fair comparison.

Figure 5(a) shows that the non-retrofitted joint (NRS) fails at a base shear level of 188 kN due to the formation of undesirable shear hinge at the joint zone. By applying a single diagonal haunch (SHRS) with 500 mm length and at an angle of 45 degrees to the beam, formation of the shear hinge is shifted from 188 kN base shear level to 248 kN (Figure 5(b)). Although the retrofitted joint can resist against a stronger earthquake with 32% higher base shear force, the joint will still fail at the joint zone first which is considered undesirable from the perspective of capacity design principle.

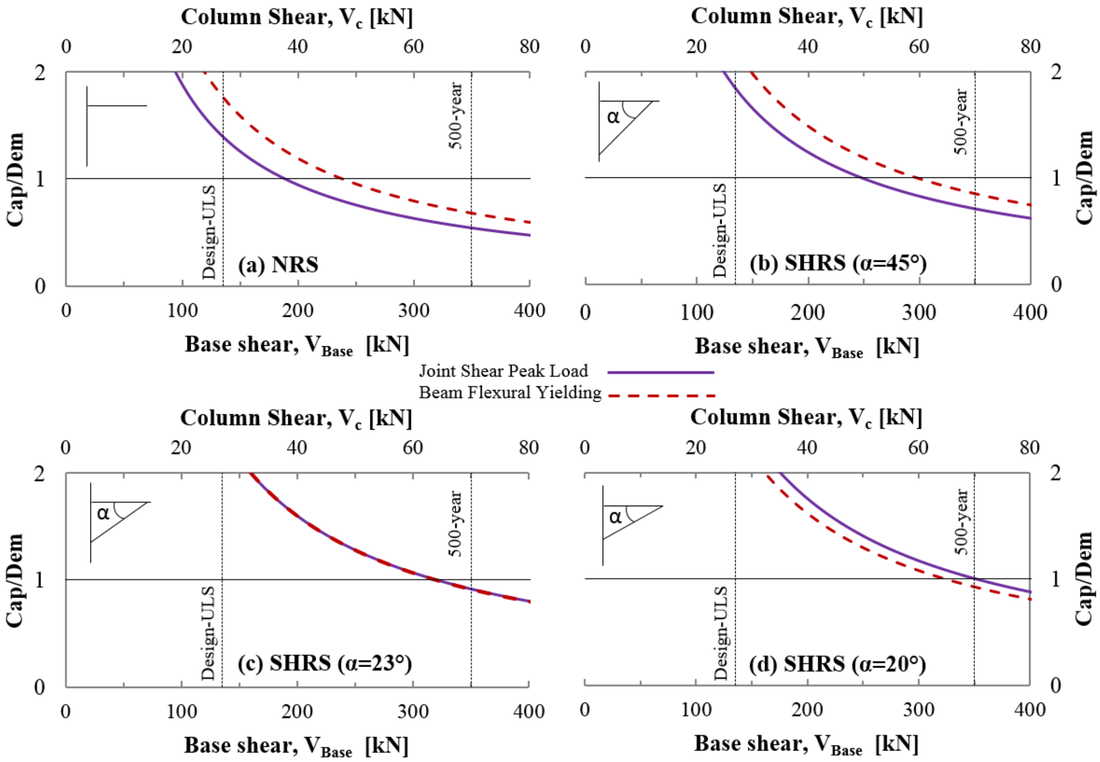

Figure 5: Joint shear and beam flexural yielding capacity-demand ratio at the exterior joint (A1) against the values of base shear force and column shear force: (a) NRS, (b) SHRS (α=45°); (c) SHRS (α=23°); and (d) SHRS (α=20°). Notes: Design ULS and 500-year indicate the shear demands at design ultimate limit state (ULS) and 500-year return period elastic response levels.

When the single diagonal haunch with the same length is oriented at a smaller angle, 23 degrees, joint shear hinging and beam plastic hinging occur simultaneously at a higher base shear level of 319 kN (Figure 5(c)). This level can be defined as the balanced scenario, whilst further angle reduction results in favourable yielding mechanism, i.e. beam flexure yielding, as well as a slightly higher capacity enhancement (Figure 5(d)). In other words, in order to achieve both benefits of single haunch retrofitting (i.e. enhancing the seismic resistance and changing the failure mechanism) at this particular beam-column joint subassembly, the angle between haunch and beam has to be smaller than 23 degrees. The key results are summarised in Table 3.

Table 3: "Failure" threshold at the exterior beam-column joint (A1)

		NRS	SHRS (α=45°)	SHRS (α=23°)	SHRS (α=20°)
Joint Shear	V_{Base}	*188 kN*	*248 kN*	*319 kN*	350 kN
Beam Flexural Yielding	V_{Base}	239 kN	296 kN	*319 kN*	*322 kN*
Location of First Failure		Joint	Joint	Joint/Beam	Beam
Capacity Enhancement		-	32%↑	70%↑	71%↑
Desirable Failure Mechanism		No	No	No	Yes

7. CONCLUSION / SUMMARY

Single diagonal haunch as a less-invasive and more architecturally favourable seismic retrofit technique has been analytically assessed based on Australian conditions. The shear demand at the joint with and without the single haunch element (Section 3), the shear transferring factor (β) as the pivotal parameter in the design of the haunch retrofitting system (Section 4), and the shear capacity at the joint (Section 5) have been presented. The angle between the haunch and the beam has been varied in order to achieve an optimal design. It was found that an angle of less than 23 degrees is required in order to enhance the earthquake resistance and to change the failure mechanism at the beam-column joint subassembly (Section 6). Further comparison with the use of double haunch elements can be found in Zabihi et al. (2016).

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