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Strengthening Design of RC Columns with Direct Fastening Steel Jackets

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Abstract: For non-seismically designed columns with insufficient strength and flexural stiffness, intense inter-story drift can be incurred during a strong earthquake event, potentially leading to the collapse of the entire building. Existing strengthening methods mainly focus on enhancing axial or flexural strength but not the flexural stiffness of columns. In response, a novel direct fastening steel jackets that can increase both flexural strength and stiffness is introduced. This novel strengthening method features straightforward installation and swift strengthening as direct fastening is used to connect steel plates together to form a steel jacketed column. This new connection method can quickly and stably connect two steel components together by driving high strength fasteners into them. In this paper, the design procedure of RC columns strengthened with this novel strengthening method is originally proposed, which includes five steps: (1) estimating lateral load capacity of damaged RC columns; (2) determining connection spacing of steel jacket; (3) estimating the lateral load capacity of strengthened RC column; (4) evaluating the axial load ratio (ALR) of strengthened RC columns; and (5) estimating effective stiffness of strengthened RC columns. Lastly, an example is presented to illustrate the application of the proposed design procedure.

Keywords: RC columns; strengthening; direct fastening; steel jackets; design procedure

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1. Introduction

Reinforced concrete (RC) moment-resisting frame buildings are widely used in schools, hospitals, and residential buildings. RC columns are the principal structural component in the resisting of lateral and gravity loads in frame buildings. Based on post-earthquake investigations [1–3], the stability of frame buildings is known to be critically dependent on the seismic performance of RC columns. The structural deficiencies identified in outmoded non-seismically designed RC columns [1–10] are (1) insufficient lap splice length of longitudinal reinforcement at column ends, (2) insufficient transverse reinforcement at the plastic hinge, (3) strong beam-weak column arrangement, (4) insufficient corrosion resistance, (5) insufficient strength due to new functional use of the building, (6) fire-induced damage, and (7) earthquake-induced damage. The first five above mentioned deficiencies may result in insufficient flexural strength, which can be mitigated by various available strengthening techniques such as RC jacketing [11,12], steel jacketing [13,14] and fiber reinforced polymer (FRP) jacketing [15–17]. However, in the case of fire or earthquake damaged columns, both flexural strength and stiffness can be reduced [18–21]. Figure 1 illustrates the seismic displacement demands of structurally damaged and non-damaged buildings within a demand spectrum. Any reduction in lateral strength and stiffness within a structural system can then induce higher seismic displacement demand during subsequent earthquake events. To restore the seismic capacity of columns to their undamaged state, both flexural stiffness and strength must be improved.

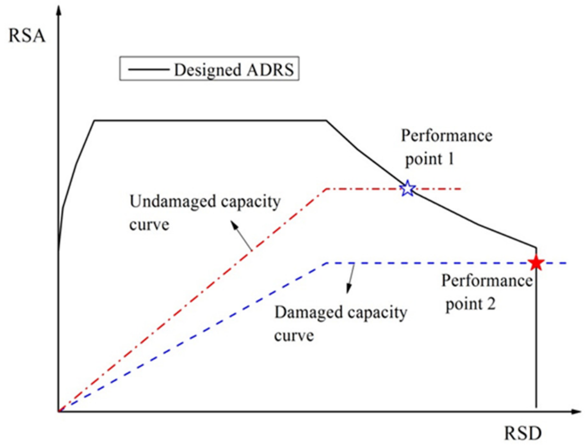


Figure 1. Design spectrum and capacity curves for damaged and undamaged structures.

In response, a novel steel jacket that deviates from traditional steel jacketing (assembled by either welding or bolting) is developed (see Figure 2). This new strengthening method offers straightforward installation and rapid strengthening, as direct fastening is used to connect steel plates together to form a steel jacketed column. The strengthening process is briefly described herein: (1) the prefabrication of steel jackets is achieved by welding the end angles to the end of the steel plates (welding can be conducted in-shop); (2) four steel plates are fastened by anchor bolts to the top and base beams; (3) the steel plates are tightly clamped to minimize gaps between them and the RC columns, and steel angles are temporarily fixed to the adjacent steel plates; and (4) the adjacent steel plates and steel angles are tightly and quickly joined by high strength nail using a powder-actuated gun (see Figure 3).

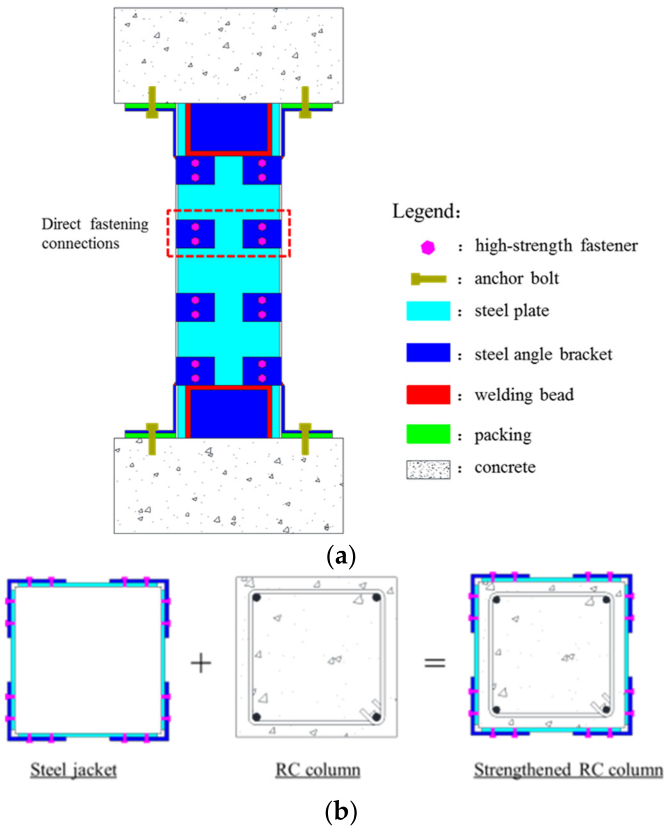


Figure 2. Proposed strengthening scheme: (**a**) front view; (**b**) plane view.

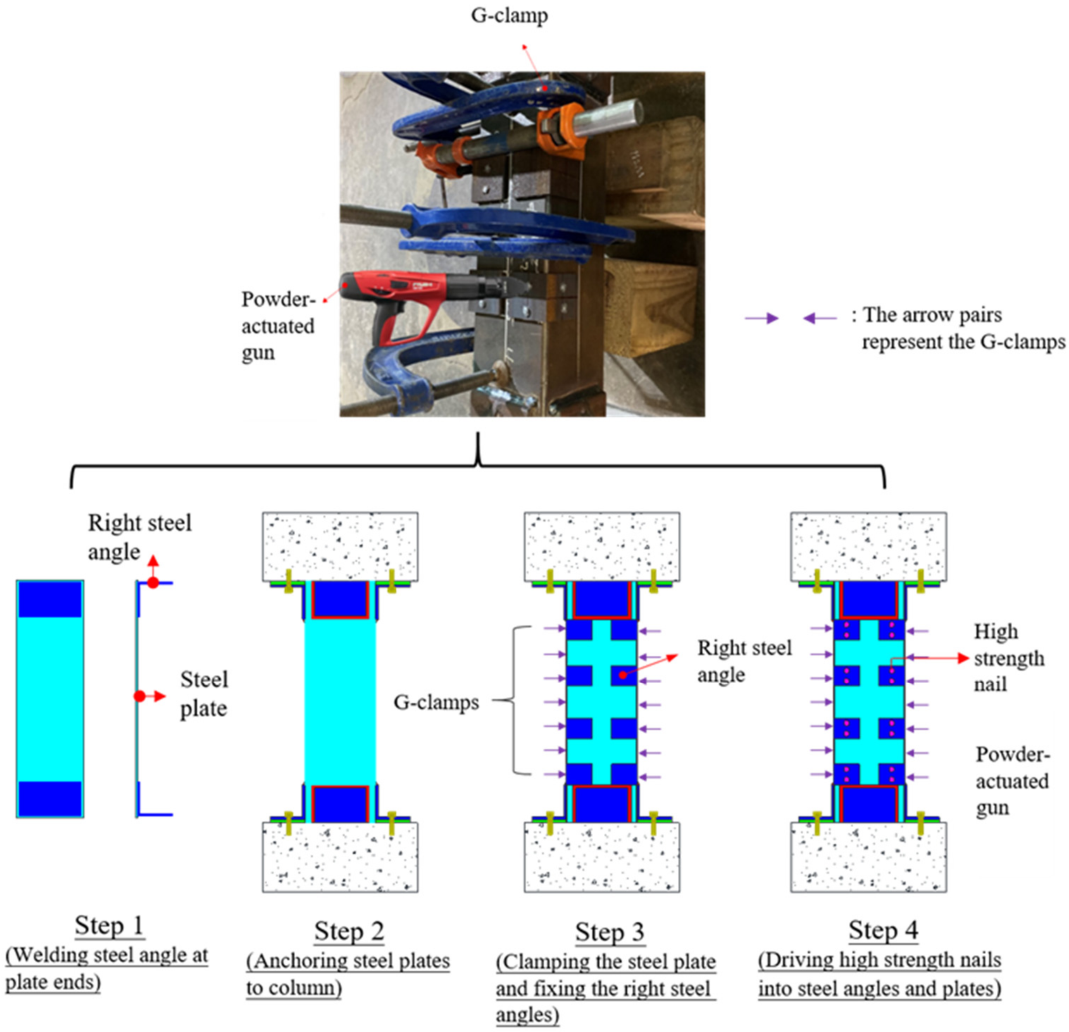


Figure 3. Strengthening procedure.

In this paper, previous experimental and theoretical studies on shear connections joined by direct fastening, axial load strengthening and the seismic strengthening of RC columns strengthened by direct fastening steel jackets are first reviewed. Based on the findings in previous studies [22–25], a design procedure of RC columns strengthened by direct fastening steel jackets is developed. To illustrate the application of the proposed design procedure, a worked example is presented.

2. Review of Previous Work

2.1. Mechanical Behavior of Shear Connectors Joined by Direct Fastening

The connections used in this novel steel jacket resist shear loading incurred as a result of concrete expansion or lateral load. The load transfer process of these connections is similar to that of single lap joints subjected to tensile force at the ends. However, the behaviors of this type of connection have rarely been studied and relevant design specifications are thus not available. Therefore, the experimentation on the single lap joints shown in Figure 4 was carried out to study the mechanical behavior of direct fastening connections [22].

Two kinds of failure modes—bearing and shear fracture—were examined in the tests. Similar to the bearing failure seen in bolted or screwed connections, this failure occurs with enlarged nail holes and bulging of the material around the fastener holes due to large plastic deformation, leading to desirable ductile behavior (see Figure 5).

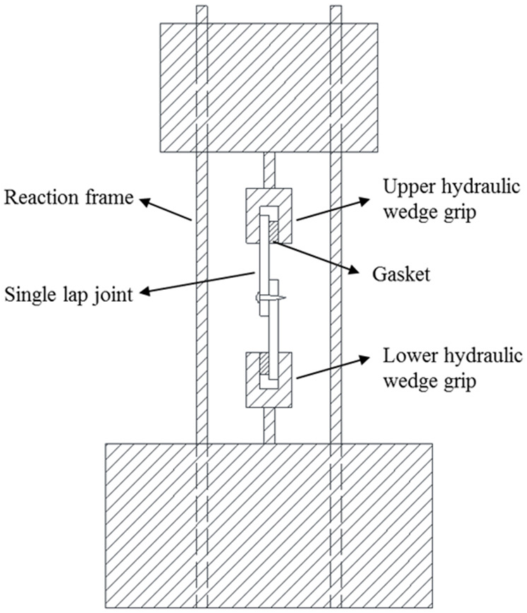
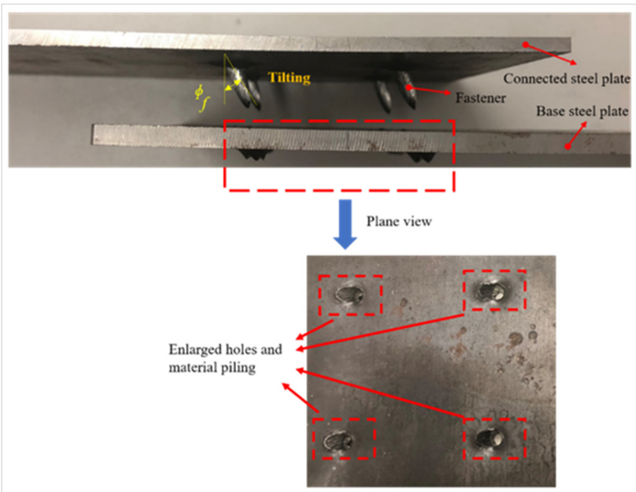


Figure 4. Illustration of connection test joined by direct fastening.



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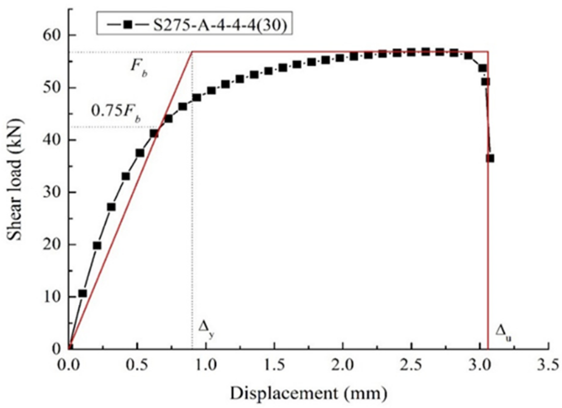


(**b**)

Figure 5. Mechanical behavior of connections: (**a**) bearing failure; (**b**) shear load versus displacement curve.

Although equations for predicting the maximum bearing resistance of bolted and screwed connections are available in current specifications EN-1993-1-8 [26], ANSI/AISC 360-10 [27], and AS 4100 [28], these equations cannot be applied to estimate the maximum bearing resistance (yield strength) of connections joined by direct fastening without modification. Therefore, a unified design equation is developed to evaluate the maximum bearing resistance of the connections:

$$F_b = \psi_{fp}\psi_{fk}\alpha_{br}d_n t_p f_{pu} \tag{1}$$

where α_{br} is the bearing resistance factor, which is taken as 1.6; ψ_{fp} is the factor that considers the effect of protuberance; ψ_{fk} is the factor that takes the effect of knurling into account; d_n is the nominal fastener diameter; t_p is the thickness of the connected steel plate; and f_{up} is the ultimate strength of the connected steel plate.

The two factors caused by the effects of protuberance and knurling are given as:

$$\psi_{fp} = \begin{cases} 1.35 & \text{without pre} - \text{drilled holes on connected steel plate} \\ 1.0 & \text{with pre} - \text{drilled holes on connected steel plate} \end{cases} \tag{2}$$

$$\psi_{fk} = \begin{cases} 1.17 & \text{fastener with knurling} \\ 1.0 & \text{fastener without knurling} \end{cases} \tag{3}$$

2.2. Axial Strengthening of RC Columns by Direct Fastening Steel Jackets

From the experimental study on the axial strengthening of RC columns by the direct fastening of steel jackets [23], the reliability and effectiveness of the proposed method were observed. Critical parameters (i.e., vertical spacing between adjacent connections, thickness of the steel plate and number of fasteners in each connection) affecting load bearing performance and deformation behavior were identified.

In the proposed strengthening method, steel plates can directly sustain axial load. The axial load contribution of the steel plates is determined by buckling strength as buckling occurs prior to the yield strength of steel plates. The buckling strength is presented in the following section.

Direct fastening connections behave in the manner of transverse reinforcement (i.e., stirrups). Due to the transverse dilation of the column, passive confinement stress is mobilized in stirrups and direct fastening connections, which enhances the strength of the column. To determine confined concrete strength, the equivalent passive confinement stress shown in Equation (3) should be determined.

$$f_{est} = \frac{m\alpha_{st} f_{yst} A_{st}}{s_{st} l_{st}} \tag{4}$$

$$f_{ed} = \frac{2\alpha_d n_f F_b}{(s_d + d_d)d_c} \tag{5}$$

where m is the stirrup legs; l_{st} is the center-to-center distance of the peripheral stirrup; s_{st} is the stirrup spacing; f_{yst} is the yield strength of the stirrup; A_{st} is the cross-sectional area of the stirrup; s_d is the clear vertical spacing of the adjacent connections; d_d is the length of the right steel angle bracket; d_c is the column width; α_{st} is the stress ratio of the stirrup; and α_d is the shear force ratio of the direct fastening connection.

A theoretical study was carried out to examine the stress ratio of the stirrup and the shear force ratio of the direct fastening connection [24]. Based on an extensive parameter study, a lower bound value of 0.34 is used to represent the stress ratio of the stirrups with a yield strength of 400 MPa. To consider the effect of the yield strength, a yield strength factor of γ_{fyst} was proposed. The yield strength factors for yield strengths of 500 MPa and 600 MPa are 0.8 and 0.66, respectively. The shear force ratio of the connections is obtained using:

$$\alpha_d = \gamma_{nf}(0.82 - 0.64\lambda_d) \tag{6}$$

where λ_d is the normalized connection spacing ($\lambda_d = s_d/d_c$); and γ_{nf} represents a factor of the number of fasteners on the shear force ratio of the connections. When the number of fasteners is 2 and 4, the factor is equal to 0.72 and 0.47, respectively.

The failure criterion proposed in [29] is used to derive the confined concrete strength of f_{cc}'. The failure criterion is given by:

$$\tau_{oct}^* = 6.9638\left(\frac{0.09 - \sigma_{oct}^*}{c - \sigma_{oct}^*}\right)^{0.9297} \tag{7}$$

$$c = 12.2445(\cos 1.5\theta)^{15} + 7.3319(\sin 1.5\theta)^2 \tag{8}$$

$$\cos\theta = \frac{\sqrt{3}}{2}\frac{s_1}{\sqrt{J_2}} \tag{9}$$

$$\tau_{oct}^* = \frac{\tau_{oct}}{f_c'} \tag{10}$$

$$\sigma_{oct}^* = \frac{\sigma_{oct}}{f_c'} \tag{11}$$

where τ_{oct} and σ_{oct} are the octahedral shear and normal stress, respectively; θ is the direction of the deviatoric stress on the deviatoric plane; s_1 is the first deviatoric stress; J_2 represents the second invariant of the deviatoric stress tensor; and f_c' is the compressive strength of the unconfined concrete.

The key parameters in the applied failure criterion are summarized as follows:

$$s_1 = \frac{f_{cc}' - f'}{3} \tag{12}$$

$$J_2 = \frac{1}{3}\left(f_{cc}' - f'\right)^2 \tag{13}$$

$$\cos\theta = \frac{f_{cc}' - f'}{3\sqrt{2}\tau_{oct}} \tag{14}$$

$$\tau_{oct} = \frac{\sqrt{2(f_{cc}' - f')}}{3} \tag{15}$$

$$\sigma_{oct} = -\frac{2f' + f_{cc}'}{3} \tag{16}$$

where f_{cc}' is the axial compressive strength of the confined concrete and f' is the equivalent passive confinement stress.

2.3. Seismic Strengthening of RC Columns by Direct Fastening Steel Jackets

To investigate the seismic performance of strengthened RC columns using direct fastening steel jackets, an experimental study was conducted [25]. Attention was given to the enhancement of flexural stiffness and strength.

On the basis of comparisons between predicted and measured effective flexural stiffness, the expressions recommended in EN 1994-1-1 [30] are advised to calculate effective flexural stiffness:

$$K_i = (EI)_s + 0.6(EI)_c \tag{17}$$

where $(EI)_s$ is the flexural stiffness provided by the steel jacketing; and $(EI)_c$ is the flexural stiffness provided by the RC column.

Buckling of the steel plate on the compressive side was observed. The corresponding buckling stress of steel plate under a compressive state should thus be embedded within the theoretical model used to predict lateral capacity. Using the Rayleigh–Ritz method and assuming the initial bowing and subsequent deflection as a trigonometric function, the buckling strength of steel plate is given as:

$$\sigma_{p,critical} = \frac{4\pi^2 D d_p}{{s_d}^2 A_p}(1-\alpha_i) \tag{18}$$

where D is the bending stiffness of the steel plate per unit width; d_p is the plate depth; E_p and μ_p are elastic modulus and Poisson's ratio of steel plate; A_p is the cross-sectional area of steel plate; and α_i is the imperfection factor.

The imperfection factor α_i is introduced to consider the initial bowing effect induced by the welding and is calibrated using the experimental results:

$$\alpha_i = 1.046 - \frac{0.73\lambda_{sr}}{100} \tag{19}$$

where $\lambda_{sr} = s_d/t_p$ is the slenderness ratio of steel plate. As the empirical imperfection factor was calibrated with limited available data, it is useful to assume a slenderness ratio of the steel plate between 14 and 39.

It is worth noting that the steel plate detached from the RC column at the bottom of the tension fiber. This was largely due to the strain incompatibility between RC column and the steel jacket on the tension side during bending. As a result, the assumption that the plane section would remain plane after deformation throughout the entire length of the column is inaccurate. Hence, a reduction factor of $\eta_i = 0.6$ was introduced for the tensile component of the steel plate when the lateral resistant capacity was estimated.

3. Design Procedure

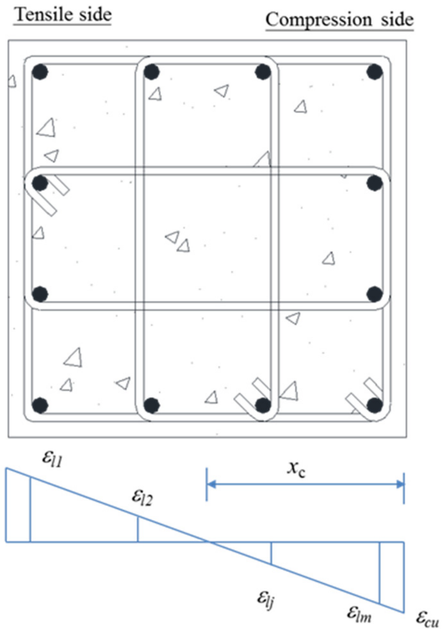
In this section, the critical parameters (i.e., steel plate thickness, fastener number and connection spacing) are first roughly determined. The feasibility of these parameters are further examined by satisfying four conditions: (1) desirable flexural failure occurs prior to brittle shear failure; (2) lateral load capacity is larger than lateral load demand; (3) ALR does not exceed the limit stipulated in current specification; and (4) effective flexural stiffness should be comparable to that of the undamaged RC column.

3.1. Estimating Lateral Load Capacity of Damaged RC Columns

The deficiency of the RC column may occur for a variety of reasons (e.g., fire). It is necessary to estimate the lateral load capacity of the damaged RC column and compare this with the load demand. According to the assumption that the plane section remains plane after deformation, the strain profile and the stress of the longitudinal rebar can be defined (Figure 6).

$$\begin{cases} -f_{yl} \le \sigma_{l1} = E_s\left(d_c - c - \phi_{st} - \frac{\phi_l}{2} - x_c\right)\frac{\varepsilon_{cu}}{x_c} \le f_{yl} \\ -f_{yl} \le \sigma_{lj} = E_s\left(d_c - c - \phi_{st} - \frac{\phi_l}{2} - (j-1)s_l - x_c\right)\frac{\varepsilon_{cu}}{x_c} \le f_{yl} \\ -f_{yl} \le \sigma_{lm} = E_s\left(d_c - c - \phi_{st} - \frac{\phi_l}{2} - (m-1)s_l - x_c\right)\frac{\varepsilon_{cu}}{x_c} \le f_{yl} \end{cases} \tag{20}$$

where s_l is the longitudinal rebar spacing; subscript lj represents the jth row longitudinal rebar counting from the tensile sides ($1 \le j \le m$); m is the total rows of longitudinal rebars which is equal to the stirrup legs; ϕ_l is the diameter of the longitudinal rebar; ϕ_{st} is the diameter of the stirrup; c is the cover thickness of the column; x_c is the compressive depth of the column; ε_{cu} is the ultimate strain of concrete and is taken as 0.003 in accordance with ACI 318 [31]; f_{yl} is the yield strength of the longitudinal reinforcement; and σ_{lj} is the stress of the jth row longitudinal rebar.



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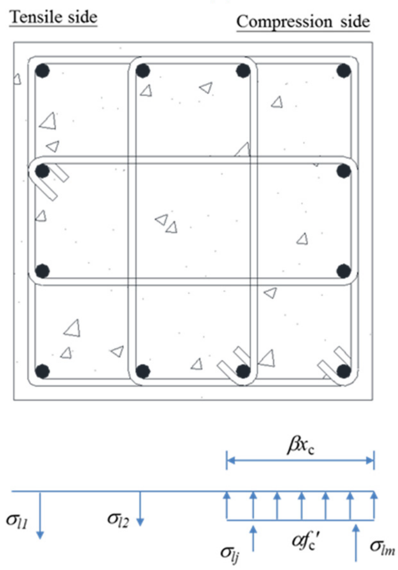


(**b**)

Figure 6. Vertical strain and stress profiles of RC column: (**a**) strain profile; (**b**) stress profile.

The stress profile of the concrete is quantified using the equivalent rectangular stress block. The compressive depth is then determined by the axial equilibrium:

$$N_0 + \frac{\sum_{j=1}^{m} k_j A_l \sigma_{lj}}{\gamma_s} - \frac{\alpha f_c' d_c \beta x_c}{\gamma_c} = 0 \tag{21}$$

where k_j is the number of the jth row longitudinal rebar; γ_s is the partial safety factor of rebar; γ_c is the partial safety factor of concrete; N_0 is the axial load; A_l is the cross-sectional area of longitudinal rebar; $f_c{}'$ is the concrete strength; and α and β are the two factors used to determine the equivalent rectangular stress block.

By taking moment to the extreme compressive fiber of the RC column, the moment capacity is given as:

$$M_{cap} = N_0 \frac{d_c}{2} + \frac{\sum\limits_{j=1}^{m} k_j A_l \sigma_{lj} (d_c - c - \phi_{st} - \frac{\phi_l}{2} - (j-1) s_l)}{\gamma_s} - \frac{\alpha f_c' d_c \beta x_c \frac{\beta x_c}{2}}{\gamma_c} \tag{22}$$

Under the assumption that the stiffness of the upper and lower joints is comparable, the lateral load resistance is then given by:

$$V_{cap} = \frac{M_{cap}}{0.5L} \tag{23}$$

where L is the length of the column.

If lateral load capacity cannot satisfy lateral load demand, the RC column can be strengthened using the direct fastening steel jackets.

3.2. Determining Connection Spacing of Steel Jacket

Flexural failure is the desirable failure mode. To ensure that desirable flexural failure occurs, the following relationship should be satisfied in light of ASCE 41-13 [32].

$$V \le V_{cap,stren} \le 0.6 V_{stren} \tag{24}$$

where V is the lateral load demand acting on the RC column; $V_{cap,stren}$ is the lateral load corresponding to the moment capacity of the strengthened RC column; and V_{stren} is the shear strength of the strengthened RC column which is determined by the following expression:

$$\begin{aligned} V_{stren} &= V_d + V_c + V_s \\ &= 0.5 \frac{d_c}{s_d + d_d} \frac{2 n_f F_b}{\gamma_s} + \frac{0.17 (1 + \frac{N_0}{14 A_c}) \sqrt{f_c'} d_c d_w}{\gamma_c} + \frac{m A_{st} f_{yst} d_w}{\gamma_s s_{st}} \end{aligned} \tag{25}$$

where n_f is the fastener number in a connection; A_c is the cross-sectional area of the column; and d_w is the depth of the column.

The first term represents the shear resistance produced from the connections. To avoid an overestimation of the shear resistance from the steel jacket, a factor of 0.5 is introduced by following EN 1998-3:2005 [33]. As the partial safety factor for direct fastening connections is absent, the partial safety factor of steel rebar is used.

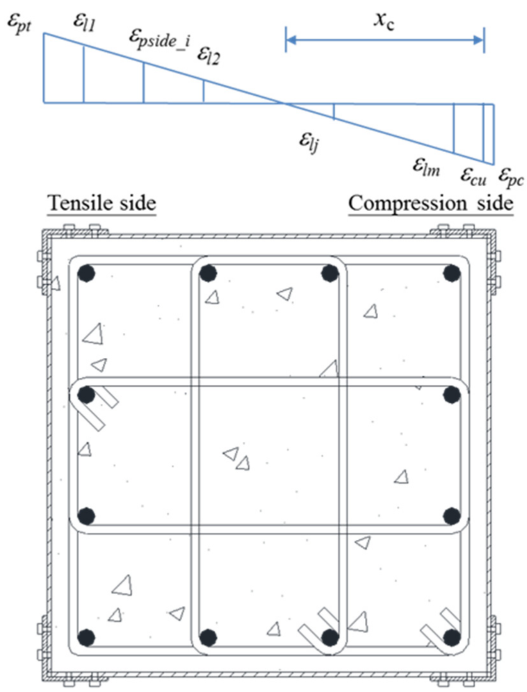
Combining the above two expressions, the connection spacing of the steel jacket can be determined.

3.3. Estimating Lateral Load Capacity of Strengthened RC Column

According to the assumption that the plane section remains plane after deformation, the strain profile and stress profile of the longitudinal rebar and steel plates of the strengthened RC column are defined, respectively (see Figure 7). It should be noted that the side plate is divided into n equal small parts to accurately define the stress in the side plate. Moreover, the buckling of the steel plate under a compressive state is embedded.

$$\begin{cases} \sigma_{pt} = E_p (d_c + \frac{t_p}{2} - x_c) \frac{\varepsilon_{cu}}{x_c} \le f_{py} \\ \sigma_{pc} = E_p (\frac{t_p}{2} + x_c) \frac{\varepsilon_{cu}}{x_c} \le \sigma_{p,critical} \\ -\sigma_{p,critical} \le \sigma_{pside\_i} = E_p (d_c - \frac{d_c - d_p}{2} - \frac{\Delta_i}{2} - (i-1)\Delta_i - x_c) \frac{\varepsilon_{cu}}{x_c} \le f_{py} \\ -f_{yl} \le \sigma_{l1} = E_s (d_c - c - \phi_{st} - \frac{\phi_l}{2} - x_c) \frac{\varepsilon_{cu}}{x_c} \le f_{yl} \\ -f_{yl} \le \sigma_{lj} = E_s (d_c - c - \phi_{st} - \frac{\phi_l}{2} - (j-1) s_l - x_c) \frac{\varepsilon_{cu}}{x_c} \le f_{yl} \\ -f_{yl} \le \sigma_{lm} = E_s (d_c - c - \phi_{st} - \frac{\phi_l}{2} - (m-1) s_l - x_c) \frac{\varepsilon_{cu}}{x_c} \le f_{yl} \end{cases} \tag{26}$$

where i represents the ith partitioned part of the steel plate parallel to the lateral load counting from the tensile side; σ_{pside_i} is the stress of the ith partitioned part of the steel plate parallel to the lateral load; Δ_i is the length of the equal small part and is equal to d_p/n; and $\sigma_{p,critical}$ is the ultimate compressive stress of the steel plate, which can be defined by Equation (8).



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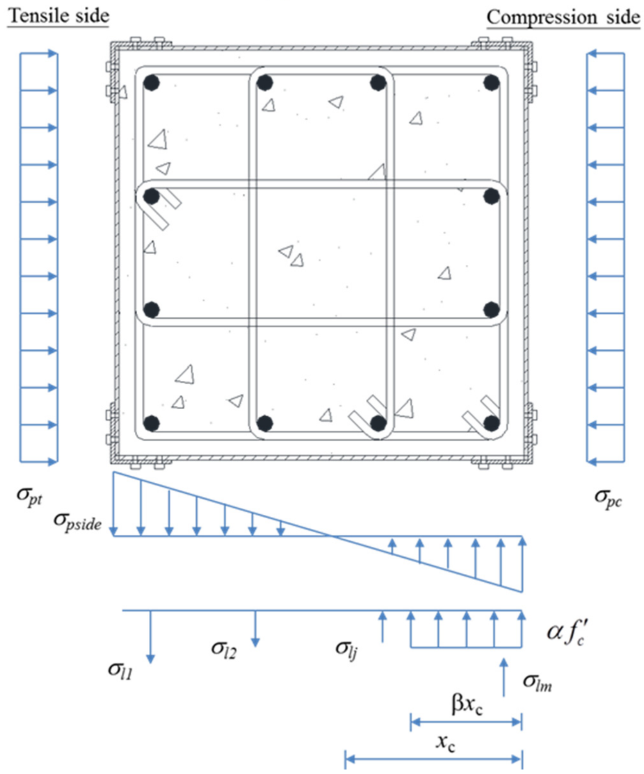


(**b**)

Figure 7. Vertical strain and stress profiles of strengthened RC column: (**a**) strain profile; (**b**) stress profile.

The compressive depth of the strengthened RC column is then determined by the axial equilibrium.

$$0 = N_0 + \frac{\sum_{j=1}^{m} k_j A_l \sigma_{lj}}{\gamma_s} + \frac{\eta_i \sigma_{pt} A_p - A_p \sigma_{pc} + 2\sum_{i=1}^{n} t_p \Delta_i \sigma_{pside\_i}}{\gamma_s} - \frac{\alpha f_c' d_c \beta x_c}{\gamma_c} \quad (27)$$

By taking moment to the extreme compressive fiber of the strengthened RC column, the moment capacity is given as:

$$\begin{aligned} M_{cap,stren} = \; & N_0 \frac{d_c}{2} - \frac{\alpha f'_c d_c \beta x_c \frac{\beta x_c}{2}}{\gamma_c} + \frac{\sum_{j=1}^{m} k_j A_l \sigma_{lj} (d_c - c - \phi_{st} - \frac{\phi_l}{2} - (j-1) s_l)}{\gamma_s} \\ & + \frac{\eta_i \sigma_{pt} A_p (d_c + \frac{t_p}{2}) + A_p \sigma_{pc} \frac{t_p}{2} + 2\sum_{i=1}^{n} t_p \Delta_i \sigma_{pside\_i} (d_c - \frac{d_c - d_p}{2} - \frac{\Delta_i}{2} - (i-1)\Delta_i)}{\gamma_s} \end{aligned} \quad (28)$$

Under the assumption the stiffness of the upper and lower joints is comparable, the lateral load resistance is then given by:

$$V_{cap,stren} = \frac{M_{cap,stren}}{0.5L} \quad (29)$$

Then, the relationship between the ultimate lateral load and the lateral load resistance of the strengthened RC column can be checked.

3.4. Estimating ALR of Strengthened RC Columns

The confined concrete divisions are shown in Figure 8a. Confined concrete 1 is confined by the erected steel jacket. Confined concrete 2 is confined by the stirrup. Confined concrete 3 is confined by the stirrup and erected steel jacket. Because the area of confined concrete 1 and confined concrete 2 is relatively smaller than that of confined concrete 3, and these two areas are close, the confined concrete divisions can be simplified in the design calculation. The simplified concrete divisions are shown in Figure 8b, in which the confined effect of confined concrete 2 is imposed on confined concrete 1. Thus, the initial three varieties of confined concrete can be equivalent to one confined concrete that is confined by a stirrup and steel jacket. For the confined concrete in Figure 8b, the imposed equivalent passive confinement stress is $f_l = f_{est} + f_{ed}$, which can be determined according to (3). Using the confined concrete strength model in (5), the confined concrete strength (f_{cc}') can be determined. Thus, the axial load capacity is given as:

$$N_{c,stren} = \frac{A_{cc} f_{cc}' + A_{c0} f_c'}{\gamma_c} + \frac{(4m-4) A_l f_{yl} + 4P_{p,critical}}{\gamma_s} \quad (30)$$

where A_{cc} is the area of confined concrete in Figure 8b; and A_{c0} is the area of the unconfined concrete in Figure 8b.

The confined concrete area A_{cc} and unconfined area A_{c0} are respectively given as:

$$A_{cc} = d_c^2 - 4\frac{d_l^2}{6} - (4m-4) A_l \quad (31)$$

$$A_{c0} = 4\frac{d_l^2}{6} \quad (32)$$

where d_l is the edge-to-edge horizontal distance of the connections, as shown in Figure 8b.

The ALR is given as:

$$\text{ALR} = \frac{N_0}{N_{c,stren}} \quad (33)$$

According to the recommendation in EN 1998-1:2004 [34], the ALR must necessarily be less than 0.65.

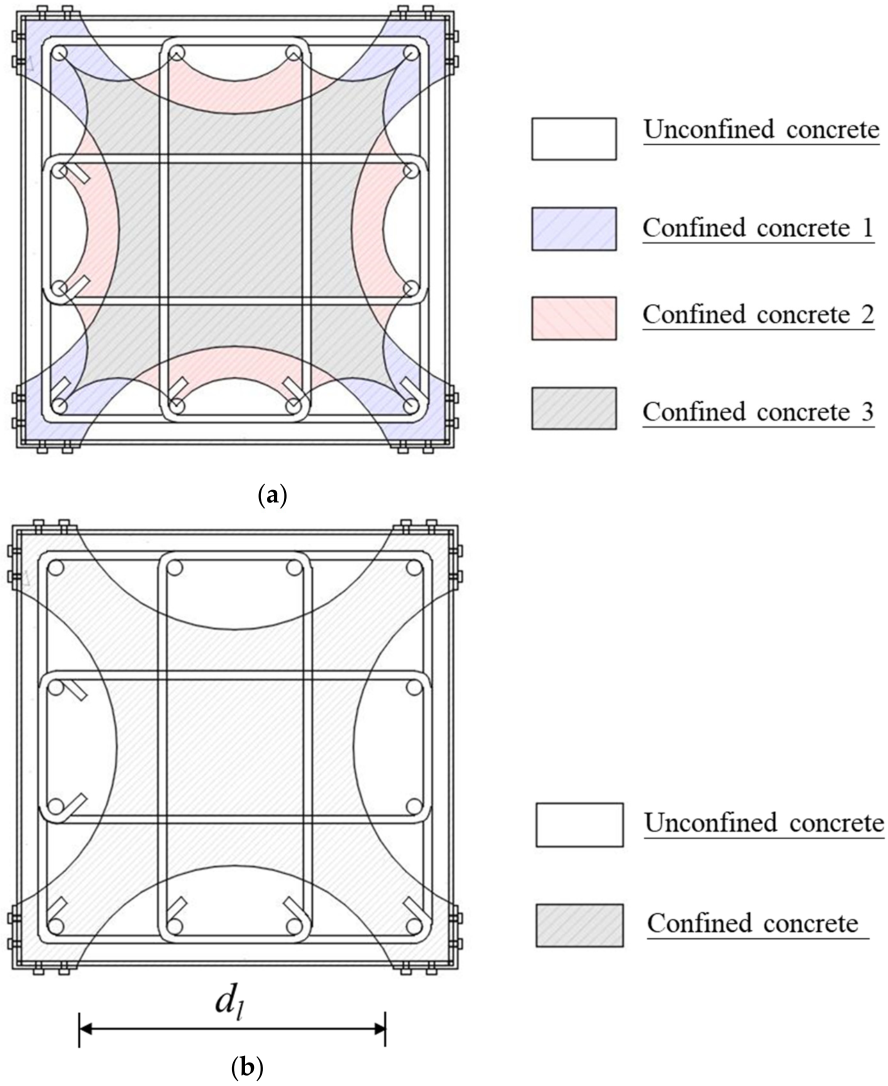


Figure 8. Divisions of the confined concrete: (**a**) exact divisions; (**b**) simplified divisions.

3.5. Estimating Effective Stiffness of Strengthened RC Columns

In light of the findings relating to the seismic strengthening of RC columns by direct fastening steel jackets, the effective flexural stiffness of the strengthened RC column can be estimated by (7). The effective flexural stiffness of the strengthened RC column should be comparable to that of the undamaged RC column, with the aim of the deformation capacity and seismic load carrying capacity remaining unchanged.

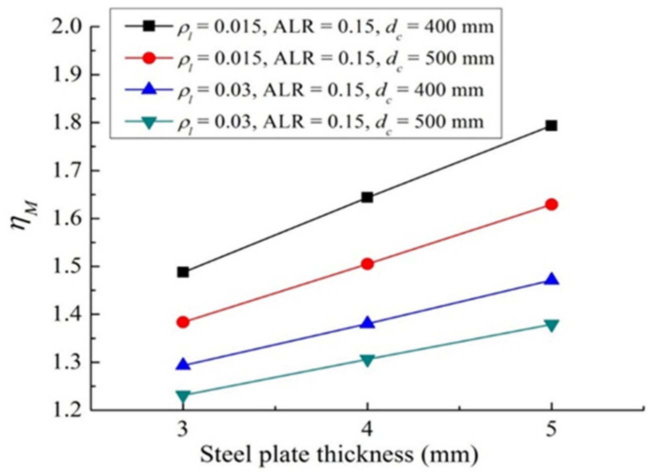
4. Parameter Study

A parameter study is conducted in this section to investigate the enhancement ratio ($\eta_M = V_{cap,stren} / V_{cap}$) of the lateral load capacity of strengthened RC columns and roughly estimate the required thickness of steel plate. Four critical variables (i.e., thickness of steel plate, cross-sectional dimension of column, longitudinal rebar ratio ρ_l and ALR) are investigated to plot the design curves. The range of these four variables is set out in Table 1. The yield strength and elastic modulus of the longitudinal rebar are 500 MPa and 200 GPa, respectively. The yield strength, ultimate strength and elastic modulus of the steel plate are 300 MPa, 400 MPa and 200 GPa, respectively. The cylinder strength of the concrete is 30 MPa. The compressive strength of the compressive steel plate is greatly affected by its slenderness ratio (λ_{sr}). For conservative consideration, λ_{sr} is taken as 38, which is the upper bound value of the slenderness ratio of steel plate determined from the experimental study [23,25].

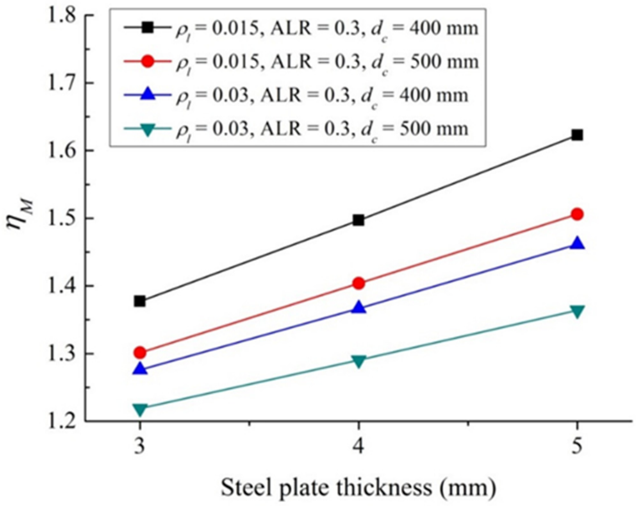
Table 1. Range of four variables in parametric study.

t_p (mm)	ALR	d_c (mm)	ρ_l
3, 4, 5	0.15, 0.3	400, 500	0.015, 0.03

It can be determined from Figure 9 that the enhancement ratio is significantly affected by the thickness of the steel plate and longitudinal rebar ratio. The enhancement ratio increases with the thickness of the steel plate, which can be seen from Equation (17). The enhancement ratio declines by 20% when the longitudinal rebar ratio increases from 0.015 to 0.03, as RC columns comprised of more longitudinal rebar possess a higher lateral load capacity, causing the decreased enhancement ratio. Compared with the effect of the longitudinal rebar ratio, the effect of the ALR on the enhancement ratio is marginal, especially for RC columns possessing higher longitudinal rebar ratios.



(**a**)



(**b**)

Figure 9. Enhancement of lateral load capacity: (**a**) ALR = 0.15; (**b**) ALR = 0.3.

5. Worked Example

The prototype of the RC column is shown in Figure 10. The height of the column is 3000 mm. The cross-sectional dimension is 500 mm × 500 mm. The reinforcement cage is formed by 12T20 and R10@150. At the ultimate limit state, the factored axial load (N_0) and factored lateral load (V) are 1400 kN and 300 kN, respectively. The elastic modulus of the rebar is 200 GPa. The yield strength of the stirrup and the longitudinal rebar is 400 MPa and 500 MPa, respectively. The cylinder strength (f_c'), cube strength (f_{cu}), and elastic modulus (E_c) of the concrete are 30 MPa, 40 MPa, and 25 GPa, respectively. The cover thickness (c) is 20 mm. It is postulated that the column experienced a fire event up to 570 °C. The strength and stiffness of the RC column can be greatly impaired after exposure to high temperature. The proposed strengthening method of directly fastening steel jackets can be used to restore fire-damaged RC columns. The elastic modulus, yield strength and ultimate strength of the steel plate are 200 GPa, 300 MPa and 400 MPa, respectively. The steel angle bracket used to connect the two adjacent steel plates measures 75 mm × 75 mm × 5 mm with a length of 50 mm. The 4 mm diameter high strength knurled fastener is used in this strengthening system. The partial safety factor of steel and concrete is taken as 1.2 and 1.5.

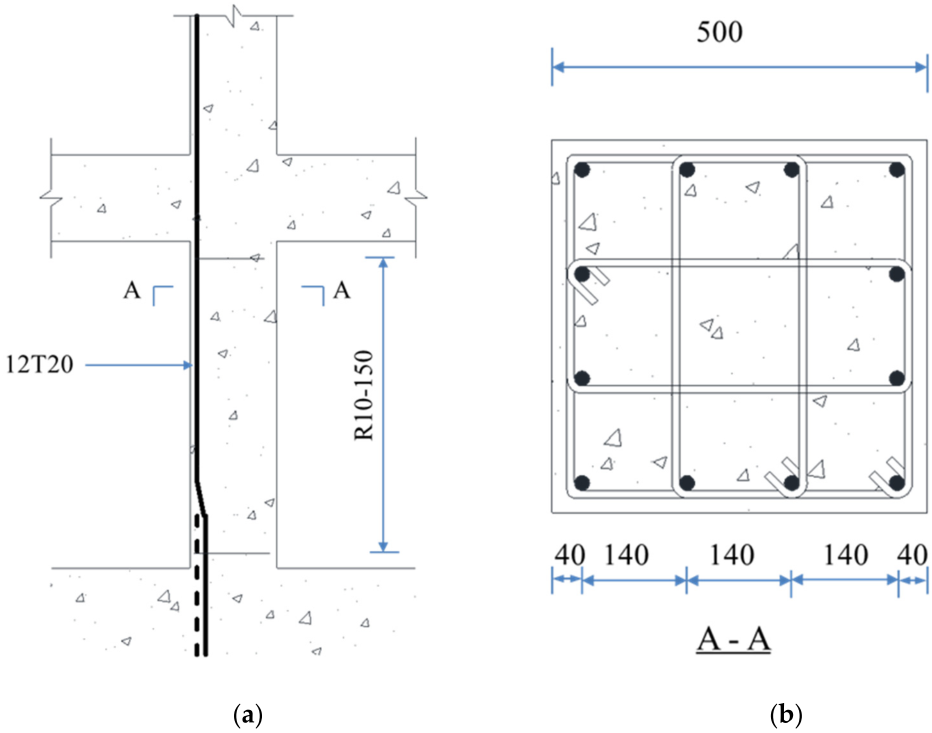


Figure 10. Reinforcement details of column: (**a**) front view; (**b**) plane view.

5.1. Evaluating Lateral Load Capacity of Fire-Damaged Column

After the concrete is exposed to fire, the concrete strength is impaired. The concrete strength post fire can be estimated according to the recommendation in EN-1992-1-2 [35]. The residual cylinder strength ($f_c'{}_{,pf}$) of the RC column following exposure to 570 °C fire is $0.5f_c'$.

According to the study on the mild steel post fire in [22], the residual strength and elastic modulus of rebar can be readily determined. The residual strength of longitudinal rebar is given as:

$$\begin{aligned} f_{yl,pf} &= f_{yl} R s_y \\ &= 500 \times \left(1.0 - 0.2 \times \tfrac{570-400}{600}\right) \\ &= 470\ \text{MPa} \end{aligned}$$

The residual strength of the stirrup is given as:

$$\begin{aligned} f_{yst,pf} &= f_{yst} Rs_y \\ &= 400 \times (1.0 - 0.2 \times \tfrac{570-400}{600}) \\ &= 376 \text{ MPa} \end{aligned}$$

The residual elastic modulus of rebar is given as:

$$\begin{aligned} E_{s,pf} &= E_s Rs_E \\ &= 200 \times (1.0 - 0.07 \times \tfrac{570-350}{500}) \\ &= 194 \text{ GPa} \end{aligned}$$

According to (20) and (21), the stresses of the four rows of longitudinal rebar and thus the compressive height of the plane section can be determined.

$$N_0 + \frac{4A_l\sigma_{l1} + 2A_l\sigma_{l2} + 2A_l\sigma_{l3} + 4A_l\sigma_{l4}}{1.2} - \frac{\alpha f'_{c,pf} d_c \beta x_c}{1.5} = 0$$

The compressive height of the plane section is solved as:

$$x_c = 304 \text{ mm}$$

The moment capacity is derived by (22).

$$\begin{aligned} M_{cap} &= N_0 \tfrac{d_c}{2} - \tfrac{\alpha f'_{c,pf} d_c \beta x_c \frac{\beta x_c}{2}}{1.5} \\ &+ \tfrac{4A_l\sigma_{l1}(d_c - c - \phi_{st} - \frac{\phi_l}{2}) + 2A_l\sigma_{l2}(d_c - c - \phi_{st} - \frac{\phi_l}{2} - s_l)}{1.2} \\ &+ \tfrac{2A_l\sigma_{l3}(d_c - c - \phi_{st} - \frac{\phi_l}{2} - 2s_l) + 4A_l\sigma_{l4}(d_c - c - \phi_{st} - \frac{\phi_l}{2} - 3s_l)}{1.2} \\ &= 311 \text{ kN} \cdot \text{m} \end{aligned}$$

The lateral load resistance of the fire-damaged RC column is given by (23).

$$V_{cap} = \frac{M_{cap}}{0.5L} = \frac{311 \times 10^3}{0.5 \times 3000} = 207 \text{ kN} < V = 300 \text{ kN}$$

Thus, the fire-damaged RC column is not safe, and the required enhancement ratio of the RC column is around 1.4.

5.2. Evaluating Lateral Load Capacity of Strengthened RC Column

Since the required enhancement ratio is around 1.4 and the longitudinal rebar ratio is 0.015, 4 mm steel plate is utilized to strengthen the fire-damaged RC column based on the findings in Figure 8. Additionally, four fasteners on the connection are used.

(a) Determining connection spacing

Flexural failure is the desirable failure mode. To realize this, Equation (24) should be satisfied.

$$300 \text{ kN} = V \leq V_{cap,stren} \leq 0.6 V_{stren}$$

According to Equation (25), the shear capacity of the strengthened fire-damaged RC column is given as:

$$\begin{aligned} V_{stren} &= V_d + V_{c,pf} + V_{s,pf} \\ &= 0.5 \tfrac{d_c}{s_d + d_d} \tfrac{2n_f F_d}{\gamma_s} + \tfrac{0.17(1 + \frac{N_0}{14A_c})\sqrt{f'_{c,pf}} d_c d_w}{\gamma_c} + \tfrac{m A_{st} f_{yst,pf} d_w}{\gamma_s s_{st}} \\ &= 0.5 \times \tfrac{500}{s'_d} \times 2 \times \tfrac{4 \times 1.6 \times 1.35 \times 1.17 \times 4 \times 4 \times 400}{1.2} + \tfrac{0.17(1 + \frac{1400 \times 1000}{14 \times 500 \times 500})}{1.5} \\ &\times \sqrt{15} \times 500 \times (500 - c - \phi_{st} - \tfrac{\phi_l}{2}) + \tfrac{4 \times \frac{3.14 \times 10^2}{4} \times 376 \times (500 - c - \phi_{st} - \frac{\phi_l}{2})}{150 \times 1.2} \end{aligned}$$

Solving the above two equations, the connection spacing is given as:

$$s_d \leq 160 \text{ mm}$$

Here, the connection spacing is taken as 100 mm. The corresponding shear capacity of the strengthened column is 623 kN.

(b) Determining ultimate compressive stress in compressive steel plate

The slenderness ratio of the steel plate is given as:

$$\lambda_{sr} = \frac{s_d}{t_p} = \frac{100}{4} = 25$$

The initial imperfection factor is given by (19).

$$\alpha_i = 1.046 - 0.0073\lambda_{sr} = 1.046 - 0.0073 \times 25 = 0.86$$

The ultimate compressive stress of the steel plate is determined by (18).

$$\begin{aligned} \sigma_{p,critical} &= \frac{4\pi^2 D d_p}{{s_d}^2 A_p}(1-\alpha_i) = \frac{4\pi^2 E_p t_p^2}{12 s_d^2 (1-\mu_p^2)}(1-\alpha_i) \\ &= \frac{4 \times 3.14^2 \times 200 \times 10^3 \times 4^2}{12 \times 100^2 \times (1-0.3^2)} \times (1-0.86) \\ &= 162 \text{ MPa} < f_{yp} = 300 \text{ MPa} \end{aligned}$$

(c) Determining lateral load capacity of strengthened RC column

The stresses in steel plate and longitudinal rebar are given by Equation (26). The compressive depth of the strengthened fire-damaged RC column is determined by Equation (27).

$$\begin{aligned} 0 = \; & N_0 + \frac{4A_l\sigma_{l1} + 2A_l\sigma_{l2} + 2A_l\sigma_{l3} + 4A_l\sigma_{l4}}{1.2} \\ & + \frac{0.6\sigma_{pt1}A_p - A_p\sigma_{pc1} + 2\sum_{i=1}^{n} t_p\Delta_i\sigma_{pside\_i}}{1.2} - \frac{\alpha f'_{c,pf} d_c \beta x_c}{1.5} \end{aligned}$$

The compressive height of the plane section of the strengthened fire-damaged RC column is solved as:

$$x_c = 302 \text{ mm}$$

The moment capacity of the strengthened fire-damaged RC column is derived by Equation (28).

$$\begin{aligned} M_{cap,stren} &= N_0 \frac{d_c}{2} - \frac{\alpha f'_{c,pf} d_c \beta x_c \frac{\beta x_c}{2}}{1.5} \\ &+ \frac{4A_l\sigma_{l1}(d_c - c - \phi_{st} - \frac{\phi_l}{2}) + 2A_l\sigma_{l2}(d_c - c - \phi_{st} - \frac{\phi_l}{2} - s_l)}{1.2} \\ &+ \frac{2A_l\sigma_{l3}(d_c - c - \phi_{st} - \frac{\phi_l}{2} - 2s_l) + 4A_l\sigma_{l4}(d_c - c - \phi_{st} - \frac{\phi_l}{2} - 3s_l)}{1.2} \\ &+ \frac{0.6\sigma_{pt1}A_p(d_c + \frac{t_p}{2}) + A_p\sigma_{pc1}\frac{t_p}{2} + 2\sum_{i=1}^{n} t_p\Delta_i\sigma_{pside\_i}(d_c - \frac{d_c - d_p}{2} - \frac{\Delta_i}{2} - (i-1)\Delta_i)}{1.2} \\ &= 530 \text{ kN} \cdot \text{m} \end{aligned}$$

The lateral load resistance of the strengthened fire-damaged RC column is given by Equation (29).

$$V_{cap,stren} = \frac{M_{cap,stren}}{0.5L} = \frac{530 \times 10^3}{0.5 \times 3000} = 353 \text{ kN} > V = 300 \text{ kN}$$

The condition in (24) is rechecked herein.

$$V_{cap,stren} = 353 \text{ kN} < 0.6V_{stren} = 374 \text{ kN}$$

5.3. Checking ALR

(a) Estimating equivalent passive confinement stress

The equivalent passive confinement stress originated from the stirrup and the connections are respectively given by (4) and (5).

$$\begin{aligned} f_{est} &= \frac{4\alpha_{st} f_{yst,pf} A_{st}}{s_{st} l_{st}} \\ &= \frac{4\times0.34\times376\times3.14\times\frac{10^2}{4}}{150\times(500-20\times2-10)} \\ &= 0.59\text{ MPa} \end{aligned} \qquad \begin{aligned} f_{ed} &= \frac{2\alpha_d n_f F_d}{(s_d+d_d)d_c} \\ &= \frac{2\times0.47\times(0.82-0.64\times\frac{100}{500})\times4\times1.6\times1.35\times1.17\times4\times4\times400}{150\times500} \\ &= 0.56\text{ MPa} \end{aligned}$$

(b) Estimating axial load capacity

For the confined concrete in Figure 7b, the imposed equivalent passive confinement stress is $f_l = f_{est} + f_{ed} = 0.59 + 0.56 = 1.15$ MPa. Using the confined concrete strength model in (7)–(16), the confined concrete strength is 22.5 MPa. Thus, the axial load capacity is given by Equations (30)–(32).

$$\begin{aligned} N_{c,stren} &= \frac{A_{cc}f'_{cc,pf}+A_{c0}f'_{c,pf}}{\gamma_c} + \frac{12A_l f_{yl,pf}+4A_p\sigma_{p,critical}}{\gamma_s} \\ &= \frac{(500\times500-12\times\frac{3.14\times20^2}{4}-\frac{2}{3}(500-70\times2)^2)\times22.5+\frac{2}{3}(500-70\times2)^2\times15}{1.5} \\ &+ \frac{12\times\frac{3.14\times20^2}{4}\times470+4\times4\times495\times162}{1.2} \\ &= 5800\text{ kN} \end{aligned}$$

The ALR is given by (33).

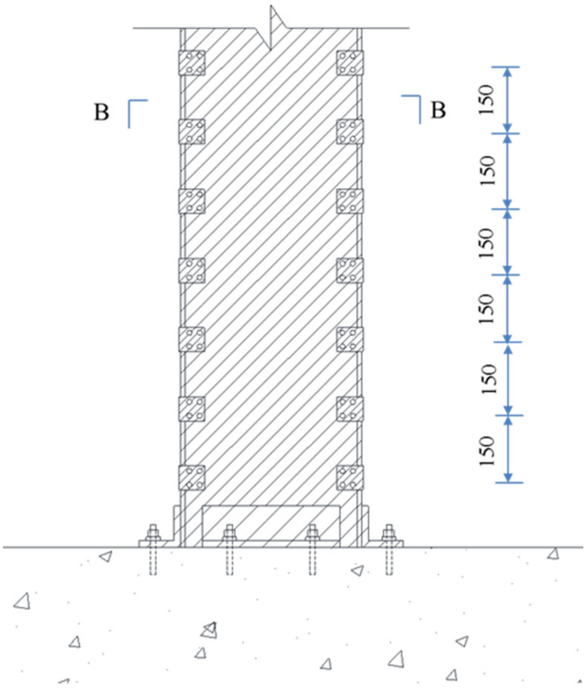
$$\text{ALR} = \frac{N_0}{N_{c,stren}} = \frac{1400}{5800} = 0.24 < 0.65$$

5.4. Checking Effective Flexural Stiffness

Based on the study conducted by Hwang et al. [6], the residual elastic modulus of concrete retains around $0.2E_c$ after exposure to a 570 °C fire event. According to (17), the effective flexural stiffness of the strengthened RC column is given as:

$$\begin{aligned} K_i &= (EI)_s + 0.6(EI)_{c,pf} \\ &= 2\frac{E_P t_p d_p^3}{12} + E_P t_p d_p \frac{d_c^2}{2} + 0.6\frac{0.2E_c d_c^4}{12} \\ &= 2\times\frac{200000\times4\times495^3}{12} + 200000\times4\times495\times\frac{500^2}{2} + 0.6\times\frac{0.2\times25000\times500^4}{12} \\ &= 8.1\times10^{13}\text{N}\cdot\text{mm}^2 \approx 7.8\times10^{13}\text{N}\cdot\text{mm}^2 = 0.6\times\frac{25000\times500^4}{12} = 0.6(EI)_c \end{aligned}$$

Until then, the parameters of the steel jacket (i.e., steel plate thickness, fastener number and connection spacing) are determined. The strengthening scheme is shown in Figure 11.



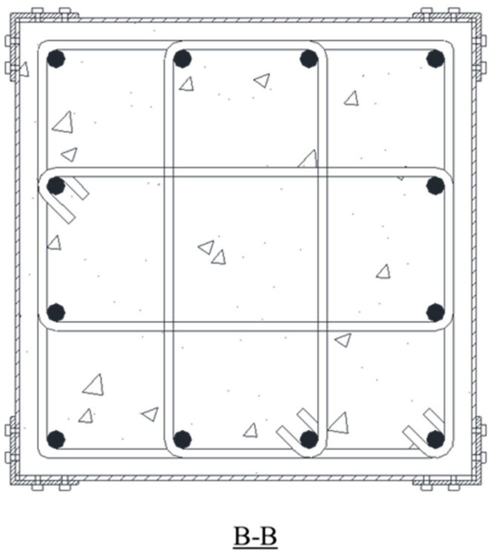


Figure 11. Strengthened RC column: (**a**) front view; (**b**) plane view.

6. Conclusions

In this paper, a novel direct fastening steel jacket is introduced. The main findings of the available experimental and theoretical studies on the proposed strengthening method are reviewed, based on which the design procedure for the proposed strengthening method is presented. The design procedure can be processed by the application of four critical steps: (1) sizing the steel plate thickness based on the required enhancement ratio and Figure 9; (2) sizing the connection spacing and fastener number by avoiding flexural failure; (3) checking lateral load demand and capacity; and (4) checking the ALR limit. If either of the final two checking conditions is not satisfied, the process should be repeated from step (1) until these two final conditions are satisfied. Subsequently, the effective flexural stiffness is examined to ensure it is similar to that of the undamaged RC column.

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