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Role of reinforcement couplers in serviceability performance of concrete members

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Abstract. Connection of reinforcing bars by couplers is a common form of reinforcement splicing. However, the variation of stiffness at the location of couplers and the potentially excessive residual slips are suspected to cause adverse impact on the serviceability, especially for structural members subjected to repeated loading. This paper studies the role of couplers in the serviceability performance of concrete members. Relevant provisions in design codes are reviewed and compared. Laboratory tests are conducted to investigate the slip behaviour of couplers. A section analysis approach based on equivalent stiffness model is proposed to account for the effects of couplers, and formulations of crack width calculation are explored for use in structural design.

1. Introduction

In precast and in-situ concrete construction, couplers are commonly used for splicing of reinforcements. Different types of couplers are available in the market, such as parallel-thread couplers, taper-thread couplers, bolted couplers, and grouted couplers [1-4]. To address the concern of possible inferiority of structural members due to reinforcement splicing by couplers, experimental investigation [5-11] and theoretical modelling [5,9,12-14] have been conducted by various researchers. Generally, proper material and construction quality control are requisite so as to ensure that the splicing region is not inferior to continuous reinforcement in terms of strength, ductility and stiffness. The relevant provisions regarding couplers in some design codes are highlighted in the following.

In the American standard AC 133: Acceptance Criteria for Mechanical Connector Systems for Steel Reinforcing Bars [15], the tensile and compressive strengths shall reach not less than 1.25 times the characteristic yield strength (f_y), and the ultimate tensile strength (f_u) is taken as the tension requirement. The same provisions are also specified in the Chinese code JGJ 107: General Technical Specification for Mechanical Splicing of Bars [16]. Similar strength requirement is adopted in Code of Practice for Structural Use of Concrete [17] in Hong Kong (CoP-HK). Ductility is monitored through the cyclic test criterion. In both AC 133 and JGJ 107, all couplers need to be tested against cyclic loads. On the other hand, couplers are classified as Type 1 and Type 2 in CoP-HK, and only Type 2 couplers need to undergo the cyclic test. Meanwhile, the determination of whether a coupler should be Type 1 or Type 2 can be ambiguous, especially when it is placed close to the beam-column junctions.

Under tension at service load, couplers may slip before yielding due to: (1) machining errors during manufacturing, threading and installation, and (2) stress concentrations caused by the abrupt changes in cross-section between the coupler and the steel bar, which can lead to local yielding. Such slip could be unrecoverable and it could give rise to residual slip after unloading. Generally speaking, the residual slip could be aggravated under repeated loading and unloading. As the residual slip is additional to the elastic deformation of the splicing region, overall there is effectively a decrease in stiffness of the splicing region at service load. Such reduction in stiffness promotes cracking and increases crack width around the coupler region. This would adversely affect the serviceability and durability of concrete members. Therefore, the stiffness should be controlled by limiting the residual slip at service load.

However, the limit on residual slip is present in some but not all design codes, and the codified slip limits are discrepant among different codes. For example, JGJ 107 stipulates a limit of residual slip in the range of 0.10 to 0.16 mm after stressed to $0.6fy$ for couplers of various sizes, while CoP-HK stipulates a limit of 0.10 mm after stressed to $0.6fy$ for all couplers. In contrast, there is no limit specified in Eurocode 2 [18] and AC 133. It is worth noting that the British Standard BS 8110 [19] once stipulated a limit of residual slip of 0.10 mm after stressed to $0.6fy$ for all couplers (i.e. the same as CoP-HK) before it was withdrawn and replaced by Eurocode 2. The reasons of adopting the above-mentioned values of residual slip limits were not clearly stated in the codes and relevant literature, though it is believed to be related to controlling the crack width. A rigorous study through experimental and theoretical investigations to determine the residual slip limits is needed.

2. Experimental investigation

Commonly used threaded couplers of different sizes were tested for their residual slip after stressing. The test set-up is depicted in Figure 1. The specimen made up of a coupler connected with grade 460 rebars at both ends was placed in the universal testing machine, which exerted loading to reach a stress level of $0.6f_y$ in the rebar, and then unloaded to zero stress. The slip was measured using an extensometer. When unloading was completed, the remaining elongations were read off from both displacement gauges of the extensometer, and the residual slip value was taken as the average of the readings. Table 1 lists the dimensions of couplers tested and the results of residual slip. For each size of couplers, 9 to 11 specimens were tested. The range of residual slip results, the average value and the standard deviation of residual slip are reported in Table 1.

Table 1. Dimensions of couplers and experimental results.

Bar size (mm)	Diameter of coupler (mm)	Length of coupler (mm)	Range of residual slip (mm)	Average residual slip (mm)	Standard deviation (mm)
16	28	40	0.00 to 0.05	0.03	0.02
20	34	48	0.00 to 0.09	0.05	0.03
25	42	60	0.00 to 0.07	0.04	0.02
32	52	72	0.06 to 0.15	0.11	0.03
40	65	90	0.00 to 0.16	0.07	0.06

In the test, the gauge length was adopted as (L + 2D), where L is the length of coupler and D is the reinforcing bar diameter. The inclusion of two times the bar diameter in the gauge length was to account for the abrupt change in cross-section at the ends of coupler, where the stress and strain distributions are not uniform until at a distance sufficiently far away from the cross-section change [13]. From Saint-Venant's principle, such distance should be approximately the transverse dimension of the element, i.e. the diameter D. It should be noted that JGJ 107 specifies a gauge length of (L + 4D) for the test, while CoP-HK does not specify the gauge length.

It can be seen from Table 1 that the residual slips varied within 0.10 mm for ≤ 25 mm bar size couplers, but the residual slips could exceed 0.10 mm for 32 and 40 mm bar size couplers. The residual slip values generally increased with the coupler size, but a mathematical relation could not be suggested in view of the scatter of results. In assessing the compliance of couplers, JGJ 107 refers to average residual slip values, whereas both CoP-HK and BS 8110 do not clearly state the basis of assessment, which may lead to practical difficulties in the enforcement of coupler quality control.

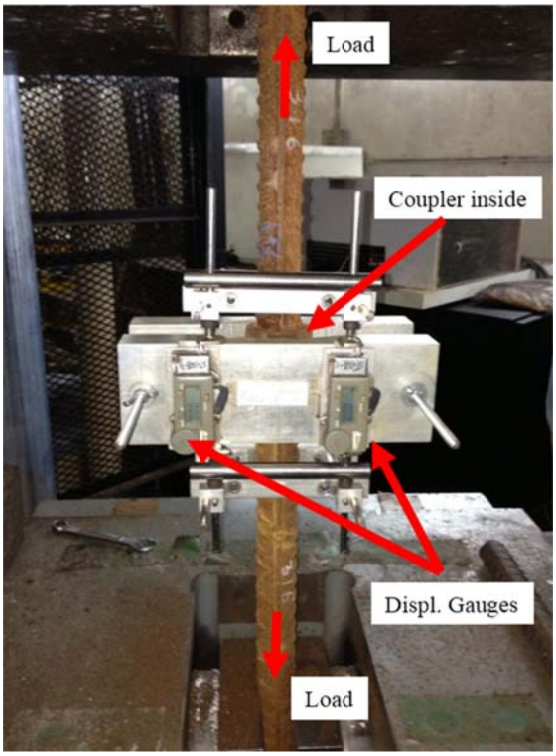


Figure 1. Test set-up.

3. Equivalent stiffness model of coupler

Ideally, the connection of rebars by a coupler does not weaken the mechanical performance in terms of strength, ductility and stiffness. Hence, it should be conservative to assume full interaction of the splicing system and to treat the connected rebars as a continuous rebar without splice. Past research [3,4,12] has shown that the stiffness of the coupler splicing region is even higher than that of a continuous reinforcement, due to the cross-section enlargement at the coupler location. Thus, the rebars connected by a coupler may be considered as a continuous rebar with an enlarged cross-section over the length of the coupler and combined with a residual slip L_{res}, as shown in Figure 2. This forms the basis for developing the equivalent stiffness model of a coupler [13].

To cater for the enlarged cross-section, herein the coupler affected region is established and is taken as (L + 2D), i.e. the gauge length adopted in the test. An equivalent stiffness is devised for modelling the slip effect in the coupler affected region. Since the coupler affected region is generally smaller than the crack spacing (for the example of 40 mm bar size, $L + 2D = 90 + 2(40) = 170$ mm compared to the crack spacing which is normally larger), there should be no concern on the necessity of sub-dividing the coupler affected region for crack width evaluation.

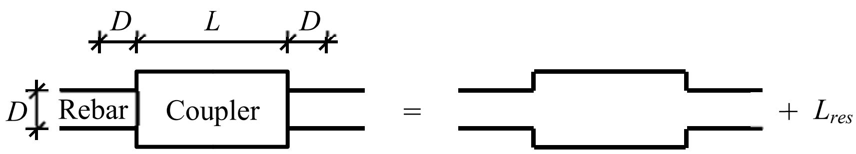


Figure 2. Modelling of coupler affected region by equivalent continuous bar.

By virtue of compatibility within the coupler affected region, the overall elongation is equal to the sum of elastic elongation of both the coupler and bar, and residual slip of the coupler. Thus,

$$\frac{0.6 f_y A_{bar}}{B_{eq}}(L+2D)=\frac{0.6 f_y A_{bar}}{E_s A_{cpl}}L+\frac{0.6 f_y}{E_s}\cdot 2D+L_{res} \tag{1}$$

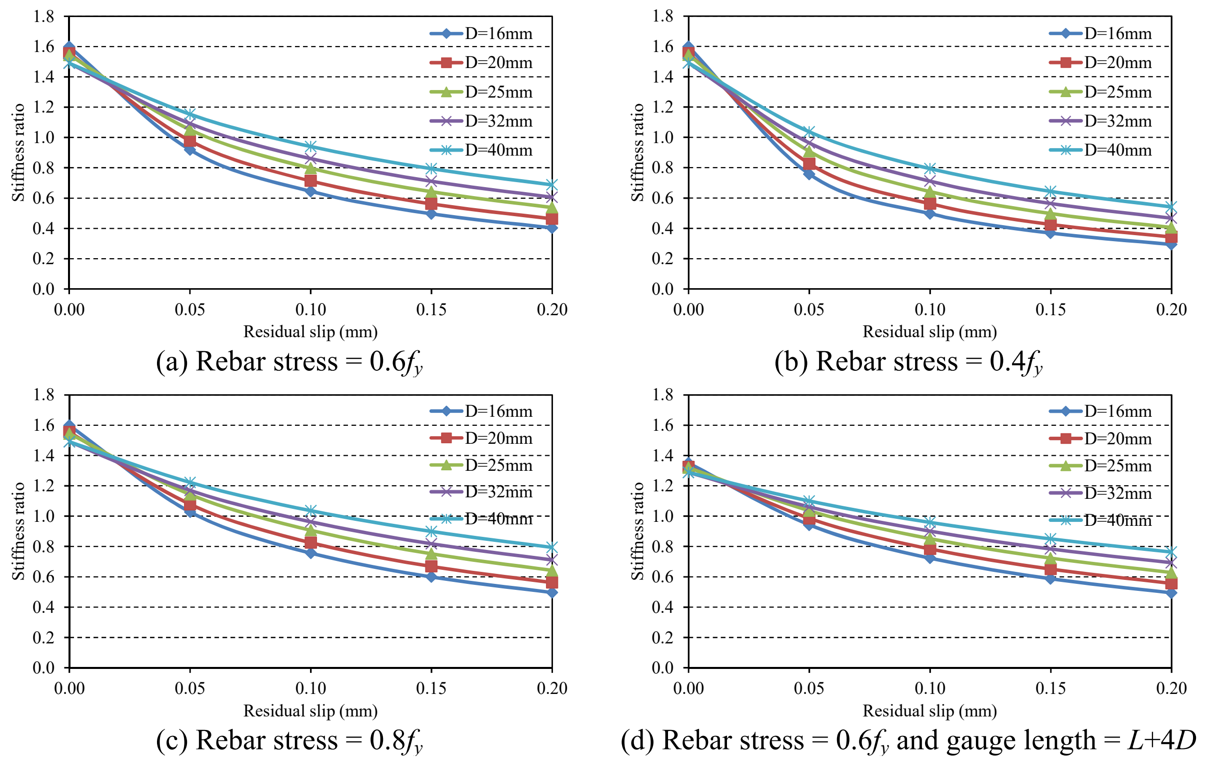
in which B_{eq} is the equivalent stiffness of the coupler affected region, A_{bar} is the cross-sectional area of the rebar, A_{cpl} is the area of solid section of the coupler, and E_s is the elastic modulus of steel. Re-arranging equation (1), B_{eq} is obtained as:

$$B_{eq}=\frac{0.6 f_y A_{bar}(L+2D)}{\dfrac{0.6 f_y A_{bar} L}{E_s A_{cpl}}+\dfrac{1.2 f_y D}{E_s}+L_{res}} \tag{2}$$

Denote the stiffness ratio between the coupler and a continuous rebar by R_{EA}, which is given by:

$$R_{EA}=\frac{B_{eq}}{0.25\pi D^2 E_s} \tag{3}$$

From equations (2) and (3), it is seen that the stiffness of coupler affected region is dependent on the residual slip, the rebar and the coupler diameters, the stress level in the rebar, and the gauge length. On the basis of these factors, a parametric study of the stiffness ratio is carried out. Figure 3 presents the variations of R_{EA}.



(a) Rebar stress = $0.6f_y$ (b) Rebar stress = $0.4f_y$

(c) Rebar stress = $0.8f_y$ (d) Rebar stress = $0.6f_y$ and gauge length = L+4D

Figure 3. Variations of stiffness ratio.

It is evident from Figure 3 that the stiffness ratio decreases with increasing residual slip. For a given residual slip, the stiffness ratio increases with the coupler size, hence it is improper to use the same residual slip limit regardless of the coupler size. Furthermore, by comparing Figures 3(a), 3(b) and 3(c), the stiffness increases with the stress level in the rebar. By comparing Figures 3(a) and 3(d), at small residual slip, the stiffness ratio decreases when the gauge length is increased, whereas at large residual slip, the stiffness ratio increases when the gauge length is increased. Therefore, apart from limiting the residual slip, the gauge length should be specified in design codes. After establishing the equivalent stiffness of the coupler affected region, the effect of couplers can be incorporated in the evaluation of serviceability performance of concrete members.

4. Coupler effects on serviceability

4.1. Section analysis

Consider a reinforced concrete section with one layer of compression reinforcement and two layers of tension reinforcement. Denote b as the width of section, h as the overall depth of section, d_p as the concrete cover to the compression reinforcement, c as the concrete cover to the tension reinforcement, x as the distance from the neutral axis to the extreme compression fibre, A_{st} as the area of tension reinforcement, and A_{sc} as the area of compression reinforcement. Further denote $\varepsilon_{st\text{-}1}$ and $\varepsilon_{st\text{-}2}$ as the strains at tension reinforcement, $\varepsilon_{cpl\text{-}1}$ and $\varepsilon_{cpl\text{-}2}$ as the strains at tension couplers, ε_{sc} as the strain at compression reinforcement, ε_{cc} as the strain at concrete extreme compression fibre, F_{cc} as the resultant compression force in concrete, F_{ct} as the resultant tension force in concrete, F_{st} as the resultant tension force in reinforcement, and F_{sc} as the resultant compression force in reinforcement, the stresses and strains distributions in the reinforced concrete section are illustrated in Figure 4.

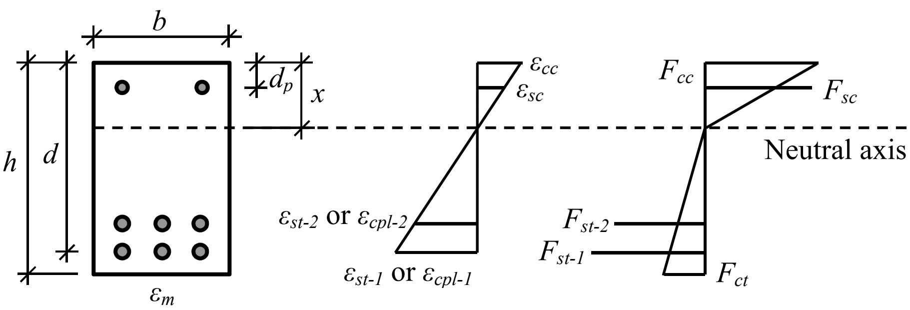


Figure 4. Stresses and strains in reinforced concrete section.

Assuming plane sections remain plane, the strain at the tensile concrete surface ε_m and the section curvature κ can be computed as:

$$\varepsilon_m = \varepsilon_{cc}(h-x)/x \tag{4}$$

$$\kappa = \varepsilon_{cc}/x \tag{5}$$

The values of ε_{cc} and x are determined from solving the force and moment equilibrium equations. For the reinforced concrete section considered, it is subjected to an external moment M_{ext}, and the external axial force is zero. Therefore,

$$\begin{cases} \sum F(\varepsilon_c, x) = 0 \\ \sum M(\varepsilon_c, x) = M_{ext} \end{cases} \tag{6}$$

At service condition, the reinforcement remains elastic. Denote the elastic modulus of reinforcement by E_s. Taking into account the structural actions of the couplers, the resultant forces in reinforcement in tension and compression are given by the following equations (the subscript i stands for the i-th reinforcement and subscript j stands for the j-th coupler):

$$F_{st} = \sum E_s \varepsilon_{st-i} A_{st-i} + \sum B_{eq-j} \varepsilon_{cpl-j} \tag{7}$$

$$F_{sc} = E_s \varepsilon_{sc} A_{sc} \tag{8}$$

Though concrete is a nonlinear material, simplifying assumptions of linear behaviour may be made. For example, in BS 8110: Part 2 [20] and CoP-HK, concrete in compression is taken to be elastic, and concrete in tension is taken to be elastic with a stress level of f_{ct} at the tension reinforcement level (f_{ct} = 1.0 MPa for short-term behavior and 0.55 MPa for long-term behaviour). Denote the elastic modulus of concrete by E_c. The resultant compression and tension forces in concrete are given by:

$$F_{cc} = 0.5 E_c \varepsilon_{cc} x b \tag{9}$$

$$F_{ct} = \frac{f_{ct}(h-x)^2 b}{2(h-x-c)} \tag{10}$$

From force equilibrium, equation (11) can be obtained:

$$0.5 E_c \varepsilon_{cc} x b + E_s \varepsilon_{sc} A_{sc} = \sum E_s \varepsilon_{st-i} A_{st-i} + \sum R_{EA-j} E_s A_{st-j} \varepsilon_{cpl-j} + \frac{f_{ct}(h-x)^2 b}{2(h-x-c)} \tag{11}$$

Taking moment about the outer layer tension reinforcement level, from moment equilibrium,

$$M_{ext} = 0.5 E_c \varepsilon_{cc} x b (d - x/3) + E_s \varepsilon_{sc} A_{sc} (d - d_p) \tag{12}$$

By resolving equations (11) and (12), the quantities ε_{cc} and x can be obtained. However, in theory, f_{ct} should not be constant but is related to the extent of cracking. In this respect, a more rigorous moment-curvature relationship needs to be employed, as discussed later.

4.2. Crack width calculation

In the prevailing practice of crack width estimation, the effect of couplers has not been specifically considered [21]. A well-known empirical formula for calculation of crack widths is provided in British Standard BS 8110: Part 2 [20] and BS 8007 [22] and CoP-HK [17], and is reproduced as follows:

$$w = \frac{3 a_{cr} \varepsilon_m}{1 + 2\left(\dfrac{a_{cr} - c_{\min}}{h - x}\right)} \tag{13}$$

where w is the design crack width at surface, a_{cr} is the distance from the point considered to the surface of the nearest longitudinal bar, and c_{min} is the minimum cover to the tension steel. The strain at the tensile concrete surface ε_m and the neutral axis distance from the compressive concrete surface x can be solved from the section analysis, in which the couplers can be represented by the equivalent stiffness model. Hence, the coupler effects can be accounted for in the crack width calculation, as also demonstrated using numerical examples [13].

On the other hand, the fib Model Code 2010 (MC 2010) [23] recommends the below formula for calculation of crack widths:

$$w = 2 l_{s\max} (\varepsilon_{sm} - \varepsilon_{cm} - \varepsilon_{cs}) \tag{14}$$

in which $l_{s\max}$ is the length over which slip between concrete and steel occurs, ε_{sm} is the average steel strain over the length $l_{s\max}$, ε_{cm} is the average concrete strain over the length $l_{s\max}$, ε_{cs} is the strain of concrete due to free shrinkage. The length $l_{s\max}$ is determined from the following equation:

$$l_{s\max} = kc + \frac{0.25 f_{ctm} D}{\tau_{bm} (A_{st} / A_{c,ef})} \tag{15}$$

where k is an empirical parameter for the concrete cover and can be taken as 1.0, f_{ctm} is the mean value of the concrete tensile strength, $A_{c,ef}$ is the effective area of concrete in tension, τ_{bm} is the mean bond strength between steel and concrete (τ_{bm} = 1.8f_{ctm} for short-term behaviour and 1.35f_{ctm} for long-term behaviour).

The MC 2010 approach of assessing crack widths is more rigorous than the empirical approach in equation (13), by considering the bond interaction between concrete and reinforcement. The strains ε_{sm} and ε_{cm} may be expressed in terms of the interpolation formula for deformation in MC 2010 and Eurocode 2.

$$\alpha = (1-\zeta)\alpha_I + \zeta\alpha_{II} \tag{16}$$

In the above, α is the deformation parameter which may be a strain, a curvature, or a rotation, α_I and α_{II} are respectively the values of deformation parameter calculated for the uncracked and fully cracked conditions, and ζ is a distribution coefficient as evaluated from equation (17) for cracked sections (ζ = 0 for uncracked sections):

$$\zeta = 1 - \beta\left(\frac{M_{cr}}{M_{ext}}\right)^2 \tag{17}$$

where β is a coefficient for the influence of duration of loading or repeated loading on the average strain (β = 1.0 for single short-term loading and 0.5 for long-term or repeated loading), and M_{cr} is the cracking moment. The respective values of ε_{sm} and ε_{cm} may be derived from the above section analysis procedure by assuming linearly elastic behaviour for steel and concrete at uncracked condition and by neglecting the tensile resistance of concrete at fully cracked condition.

Equation (16) enables the rational simulation of deformation characteristics of a cracked reinforced concrete member, where partial interaction between reinforcement and concrete develops. Upon cracking, the moment-curvature relationship becomes nonlinear, and the curvature may be adopted as the deformation parameter to yield the below interpolation formula:

$$\kappa = (1-\zeta)\kappa_I + \zeta\kappa_{II} \tag{18}$$

The curvature values are computed based on equation (5). In so doing, κ_I is evaluated from section analysis with the tensile capacity of concrete taken in account, whereas κ_{II} is evaluated from section analysis with the tensile resistance of concrete neglected.

5. Conclusions

The role of couplers in the serviceability performance of reinforced concrete members has been investigated. Though testing coupler specimens of different sizes, it has been found that the residual slip is related to the coupler size, the applied stress level, and the gauge length. Therefore, the rebar stress level and the gauge length for testing of residual slip should be specified in design codes, and the residual slip limits for individual coupler sizes should be stipulated. The equivalent stiffness model has been developed for physical modelling of couplers. A parametric study has revealed the dependence of equivalent stiffness of coupler affected region and stiffness ratio on the residual slip, coupler size, stress level in the rebar, and gauge length. For adoption in serviceability design, formulations to account for the coupler effects in section analysis and crack width calculations have been derived.

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