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Numerical study of retrofitted deep coupling beams by bolting restrained steel plate

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Summary

Deep reinforced concrete (RC) coupling beams with low shear span ratios and conventionally reinforced shear stirrups tend to fail in a brittle way with limited ductility and deformability under reversed cyclic loading. Experimental studies have shown that bolting restrained steel plate (BRSP) to existing deep RC coupling beams can enhance the deformability and energy dissipation while maintaining the flexural stiffness, improving the beams' performance during an earthquake. In this study, a nonlinear finite element package ATENA was used to simulate the overall behavior of three previously tested BRSP retrofitted coupling beams. This paper presents the numerical study for the accurate simulation of BRSP deep coupling beams. In order to obtain the accurate results, the effect of plate buckling is considered in the models. Finally, the simulation model was validated by the available experimental results.

Keywords: Deep coupling Beams, Seismic retrofitting, Plate Buckling, Finite Element Model

Introduction

Many old reinforced concrete (RC) buildings in developed countries need to be strengthened due to the aging of construction materials, change in functional use or new design loading requirements. Coupled beams in coupled shear walls are very important structural components that provide the necessary lateral strength, stiffness and deformability for the whole building to resist extreme environmental loads, including wind and earthquake. To ensure the desired behaviour of coupled shear wall systems, coupling beams should be sufficiently strong for resisting wind load, and have good energy dissipation ability and low strength degradation rate for seismic resistant applications. While existing deep coupling beams inevitably failed in diagonal tension when the shear reinforcement was insufficient (Paulay, 1971). In past decades, the design of many concrete buildings (including coupling beams) in China and Hong Kong did not take into account earthquake actions. According to the new design codes, many existing coupling beams are classified to be deficient in shear. Sudden failure of these coupling beams will threaten the structural safety of the buildings.

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Compared with the research about strengthening of existing RC floor beams, only a few studies are related to strengthen of existing RC coupling beams. Harries et al. (1996) studied a shear strengthening method for existing coupling beams with a span-to-depth ratio of 3.0. In their study, the retrofitting measures involved a number of different attachment methods to fix the steel plate to one side of the coupling beams. They found that the composite method of bolting with epoxy bonding to attach the steel plates both in the span and at the ends performed the best. Su and Zhu (2005) studied a shear strengthening method for RC coupling beams with a span-to-depth ratio of 2.5. To strengthen the coupling beams they bolted steel plates to two ends of wall panels without adhesive bonding. They conducted experimental and numerical studies to prove that this retrofitting method could greatly increase the shear capacity of medium length coupling beams. Bolt slipping effect was considered in their numerical model. In all their experiments, minor buckling at the steel plate was observed but the influence of local buckling on the behavior of composite coupling beams was not investigated. Most of the previous studies focused on coupling beams with span-to-depth ratios larger than 2.0. Since the widths of the door and window openings usually range from 1.0 to 1.5m, most coupling beams above doors and corridors are quite short and deep. Large-scale experiments on the BRSP deep coupling beams with the span-to-depth ratios 1.1 have been carried out. The experimental study reveals that the deformation and energy dissipation of BRSP deep RC coupling beams improved while the flexural stiffness was not increased. Moreover, by using lateral restrained steel plates, the specimens had better post-peak behavior, a more ductile failure mode and better deformability.

In this study, an accurate and efficient nonlinear finite element analysis (NLFEA) using the finite element program ATENA (2000) was conducted to investigate the whole behaviour of the BRSP deep coupling beams such as the load-rotation curves and failure characteristics. In order to get accurate results, plate buckling effect was considered in the numerical model. Finally, the numerical model was validated by the experimental results.

Proposed Retrofitting Method

Details of Test Specimens

Test setup and loading application procedure of the experimental study can be found in Su and Cheng (Accepted). Three specimens with the same dimensions and reinforcement specifications as shown in Fig.1, but different retrofitting schemes, were fabricated and tested. The sizes of coupling beams were 450 mm deep by 120 mm wide with a clear span of 500 mm and with a span-to-depth ratio of 1.1. The top and bottom longitudinal reinforcements of the coupling beams were of four 12 mm diameter high-yield reinforcement bars with a steel ratio of 0.8%, and the side bars included four 8 mm diameter mild steel round bars. The shear reinforcements of the coupling beams consisted of four 8 mm diameter hoops with a 125 mm pitch. This shear reinforcement arrangement was selected to represent the shear-deficient coupling beams in old existing buildings.

The first specimen DCB1 with a plain RC arrangement was used for control purposes. Specimens DCB5 and DCB6 were retrofitted with 3 mm and 4.5 mm grade 50 steel plates, respectively. A buckling control device (as shown in Fig.2), was mounted onto the beam span of Specimens DCB5 and DCB6. By providing two steel angles (L70×70×5mm) along the top and bottom edges of the steel plate, the possible lateral buckling of the steel plate in the span at the edges was suppressed. The steel angles were designed in accordance with the provisions of web stiffener design of steel beams given in BS5950 (BSI 1990). To avoid adding extra strength and stiffness to the composite coupling beam, the lateral stiffeners were connected to a steel plate by four bolt connections with slotted holes, which allowed the two lateral stiffeners to freely rotate and move in the longitudinal direction.

The anchorage at the ends of the steel plates was designed to be strong enough to transfer all of the forces from the steel plates to the wall anchors. To avoid buckling in the anchorage regions, thicker steel plates, measuring 6 mm and 8 mm thick, were added to DCB5 and DCB6, respectively, at the anchorage regions. To minimize any possible slippage between the various components at the connections, dynamic set washers developed by HILTI Corporation were used; the advantage of these washers is that bolt-slip can be minimized by injecting adhesive to fill up the gaps between the bolt shank and surrounding concrete.

Experimental results

Strength, Deformation and Ductility

Table 1 shows that the attached steel plates increased both the ultimate capacity and deformability of the coupling beams. The shear strengths V_{max} of DCB5 and DCB6 were increased by 41% and 50% while the ultimate rotations θ_{max} were increased by 82% and 190%, respectively. These results show that by adding a thicker steel plate, the increase in the rotation deformability is much higher than the increase in the strength. The BRSP deep RC coupling beams can easily withstand a chord rotation demand of 2% or more induced by a large earthquake.

By comparing the test results of the three specimens, it can be concluded that this new seismic retrofitting method can increase the deformation of deep RC coupling beams but not their flexural stiffness. Since the total amount of base shear induced in a building during an earthquake is dependent on the lateral stiffness of the structure, this method would not change the seismic base shear acting on the

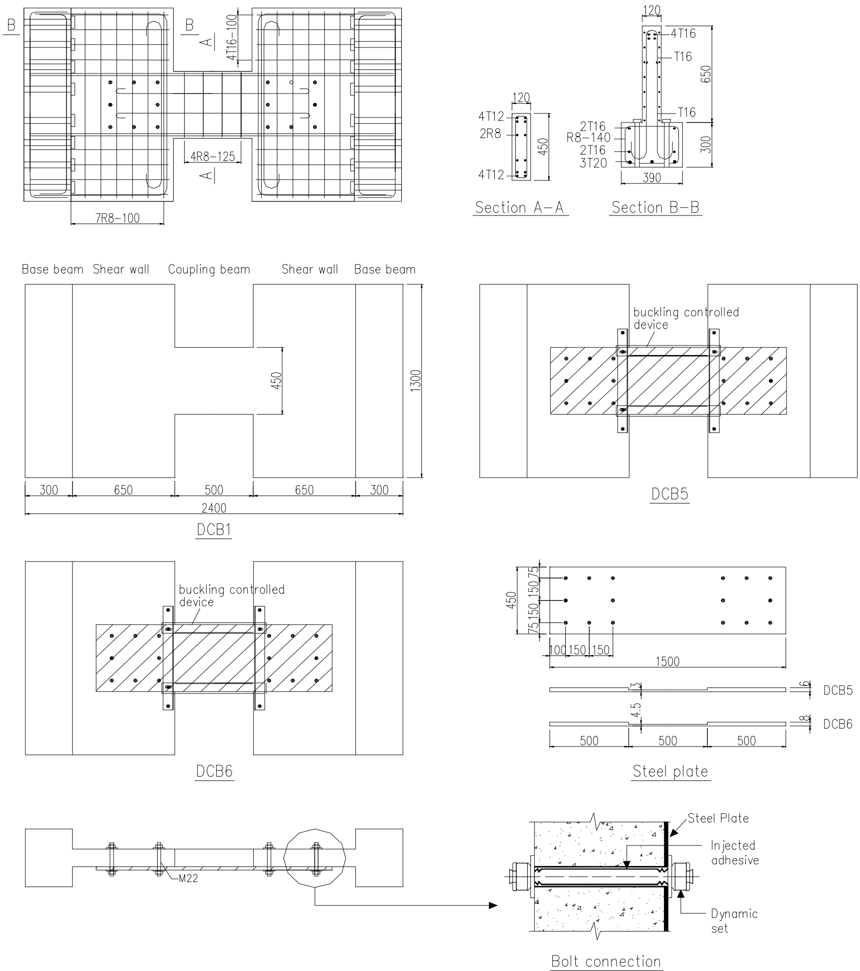


Figure 1: Details of test specimens

building.

Crack Patterns and Failure Behaviors

The concrete crack patterns of the test specimens were similar. The extensive diagonal cracks indicate that the shear capacity of the beams was insufficient. Compared with the peak strengths of the specimens, the strength did not change much by adding restrained steel plates with thicknesses of 3 mm to 4.5 mm. This indicates that the capacity of the concrete dictated the peak strength of the composite coupling beams. When the shear links yielded, the strength could not increase much further and shear link yielding caused the crack width to increase at a faster rate. When almost the entire beam was cracked, mainly in the diagonal direction,

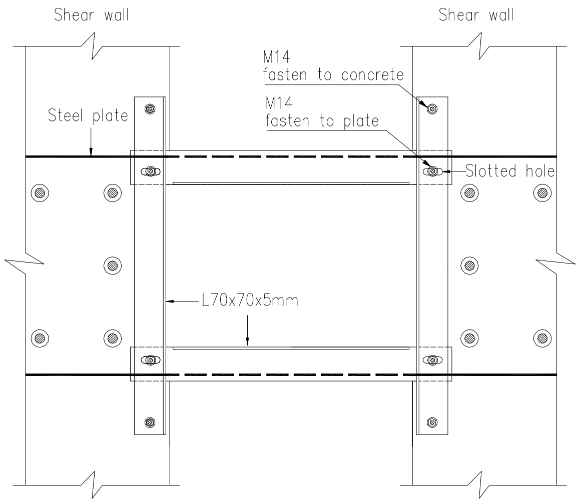


Figure 2: Buckling controlled device

concrete crushing started to occur at the beam-wall joints and the concrete spalled at the compression corners. Then, local buckling of the steel plate started to occur at these locations. Plate buckling shapes of the specimens are depicted in Fig. 3. With added buckling control, the plate buckling at the beam-wall joints was suppressed and a continuous shear transfer medium across the joints was provided. The steel plate could continue to take a larger share of the load at the post-peak region, alleviating concrete crushing at the compression region. This can explain why the failure modes of DCB5 and DCB6 were more ductile than that of DCB1.

Table 1: Summary of experimental results

Specimen	Failure Mode	V_{max} (kN)	V_{max}% increased	θ_{max} (rad)	θ_{max} % increased	K_o (kN/mm)
DCB1	Very brittle	238	N/A	0.011	N/A	92
DCB5	Ductile	335	41%	0.02	82%	93
DCB6	More ductile	356	50%	0.032	190%	66

Numerical Study

Experimental study has revealed that this retrofitting method has many advantages for existing deep coupling beam especially in the large seismic regions. However, due to the complicated composite actions among the concrete coupling beam, steel plate and bolt group, available structure codes cannot be used for the design of this BRSP coupling beams. The plate buckling has a great effect on the whole behavior of the composite beams. The prediction of the shear capacity of retrofitted coupling beams based on the full plastic section assumption without considering the effects

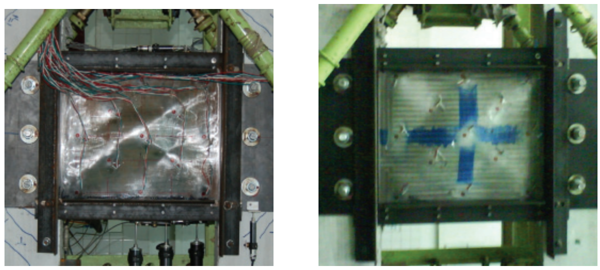


Figure 3: Plate buckling for DCB5 and DCB6 (R)

of plate buckling would overestimate the true capacity. In order to accurately simulate the behavior of BRSP beams, a nonlinear study was carried out to evaluate the ultimate strength of steel plates. Then a nonlinear finite element package ATENA (2000) was conducted to simulate the overall behavior of BRSP coupling beams with consideration of plate buckling effects in the modeling.

Ultimate strength of steel plate

General

The main purpose of this section is to evaluate the ultimate stress (σ_u) of steel plate with the consideration of geometric and material non-linearity. The main parameters such as the plate's aspect ratio, initial imperfection, boundary conditions and material properties were considered. In this study, the finite element software ABAQUS 6.8 was utilized to calculate the strength of steel plate.

Two types of analyses were carried out. Firstly, an eigenvalue buckling analysis was conducted to obtain the critical buckling stresses and the corresponding buckling modes. This analysis provides guidance to assess the reliability of the mesh, load and boundary conditions assumed for the model. Secondly, a geometric and material nonlinear analysis, of which the finite element mesh and boundary conditions are the same as that of the eigenvalue analysis, was carried out. The initial imperfection of steel plate was presented in the form of initial out-of-plane deflections. In the present study, the first local buckling mode was chosen as the form of initial deflections. The maximum magnitude of initial geometric imperfections at the plate was taken as 0.003b (b=depth of the steel plate), as suggested by Wright (1993). The yield stress of steel plate is 360MPa, elastic modulus is 200000MPa and Poisson's ratio is 0.3.

Boundary Conditions

The steel plate was restricted to unilaterally buckle between bolt connectors. To simulate this situation, the structural model consisted of a single plate laterally restrained along the dash line as shown in Fig.4. Furthermore, contact elements were added between the steel plate and concrete to ensure the plate buckled unilaterally.

To restrain the possible rigid body modes and to maintain the numerical stability, the degrees of freedom of the line near the centre of steel plate were fixed.

The tensile stress σ_x, in-plane shear stress τ_{xy} and the in-plane moment M_x coexisted in the steel plate to form an equivalent force field as shown in Fig. 4. The relation between σ_x andM_x can be easily obtained by using equivalent force calculations. A ratio of tensile stress to shear stress (α=σ_x/τ_{xy}) was defined to represent the relative intensity between the tensile stress and the shear stress. Moreover, as the distribution of the stresses along the force boundary was also unknown, transition elements with a larger elastic modulus were adopted to distribute the applied forces to the steel plate. In this way, the numerical results obtained are found to be in good agreement with the experimental results.

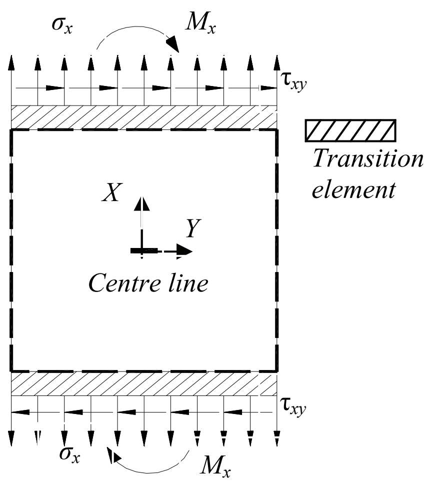
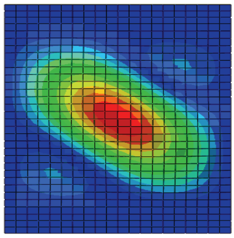
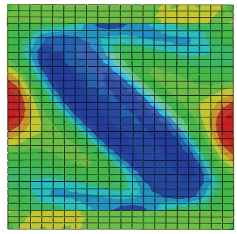


Figure 4: Boundary condition

Design model of strength

Fig. 5 shows the first buckling mode and shear stress distribution of the steel plate. As expected, the maximum out-of-plane deflection occurs at the center of the plate. Fig. 6 demonstrates the effects of the ratio of tensile stress to shear stress (α= σ_x/τ_{xy}) on the ultimate stress of steel plate. σ_0 is the ultimate stress of steel plate without the consideration of geometric non-linearity. It is found that the presence of tensile stress σ_x can increase the ultimate stress. This is due to the reason that local buckling effect can be alleviated with the presence of tensile stress field. In this study, we assume that the ratio of tensile stress to shear stress cannot be larger than 2.0.





First buckling mode shear stress distribution

Figure 5: Behavior of steel plate

Modeling of the composite beam

Modeling of concrete material

The constitutive model of concrete element SBETA is based on the smeared material approach with the consideration of non-linear elasticity and fracture mechanics. The model is described by the stress-strain curve (as shown in Fig. 7). This concrete material model has considered the following factors (1) nonlinear behavior in compression including hardening and softening, (2) fracture of concrete in tension based on nonlinear fracture mechanics, (3) biaxial strength failure criterion, (4) reduction of compression after cracking and (5) reduction of the shear stiffness after cracking.

The material parameters have been determined from the test results of local concrete.

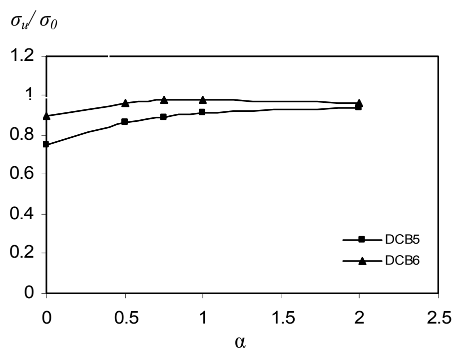


Figure 6: Strength model of steel plate

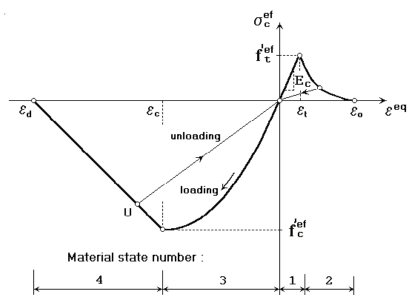


Figure 7: Concrete model

Modeling of steel plate considering plate buckling effect

The external steel plate was modeled as an elasto-plastic isotropic hardening material with the behavior similar in compression and tension. Assuming the ratio of tensile stress to shear stress was 0.5, the ultimate stress of steel was reduced according to the relationship presented in Fig.6 to account for the plate buckling effect.

Verification of finite element model

The concrete was modeled by four-node SBETA element. The element size of 20mm was adopted for the steel plate region. Two layers of coincident nodes were defined. The first layer of nodes was used for defining reinforced concrete elements and the second layer of nodes was used for steel plate elements. The steel plate elements, which were modeled by four-node elements of the same size, were connected with reinforced concrete elements by the anchor bolt elements. Smeared reinforcement models were used for the distributed bars, where the perfect bond between concrete and steel was assumed. The beam longitudinal reinforcement was modeled by two-node discrete bar elements. Fig.8 shows the composite numerical model.

In order to simulate the external steel plate, the control specimen without steel plate and bolt connections was firstly modeled by NLFEA. The results were then compared with the experimental results to validate the material parameters of concrete and steel bars. Then finite elements related to the steel plate were then added to the model. Fig.9 shows the load-rotation curves obtained form the NLFEA and test. The dash line demonstrates the results without considering plate buckling effect. It is observed that if plate buckling effect was not considered, the result always overestimate the strength of the BRSP coupling beams.

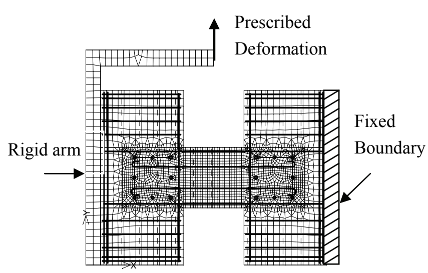


Figure 8: Finite element model

Conclusions

This study confirmed that the present NLFEA accurately predicted the load-rotation curves of BRSP coupling beams. In order to yield accurate results, plate buckling effect has to be considered in the finite element model. The new model is simple, computationally efficient and able to capture the overall behavior. Thus this model can be used for further studies of BRSP deep coupling beams and be instrumental in

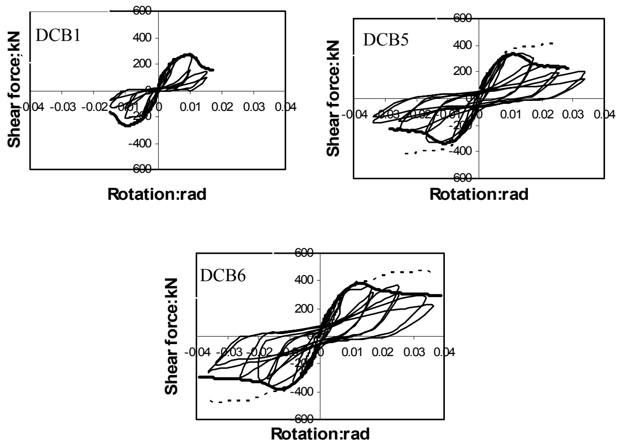


Figure 9: Experimental and numerical load-rotation curves

conducting detailed parametric studies in order to establish the design methodology for practical applications.

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