Experimental and Numerical Studies on Steel-Encased Overlapping Column Transfer Assemblies

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Abstract

Force transfer in misaligned columns in building structures can be implemented using overlapping column transfer assemblies. Two 1/6 scaled models of overlapping column transfer assemblies were tested to investigate their mechanical behavior under monotonic loadings. Both specimens were constructed using reinforced concrete and had an overlapping ratio of 0.56, one of which was encased with a steel truss with a cross-sectional steel plate ratio of 3.73%. A finite element method-based model was built, verified and used for parametric studies. Results indicated that the load bearing capacities of the steel-encased assembly increased by 35% compared to that of the assembly without steel. The increase in cross-sectional steel plate ratios and decrease in overlapping ratios improved the load bearing capacities of steel-encased assemblies.

Keywords: overlapping column transfer assembly; transfer block; steel truss; experimental study; numerical simulation

1. Introduction

In the presence of misaligned columns in multi-storey or high-rise buildings, the storey transfer technique is typically applied to implement a vertical force transfer. Fig.1 illustrates the commonly used storey transfer technique including the application of a transfer girder (Lee et al., 2011), transfer slab, and transfer truss. In the past ten years, overlapping column transfer assemblies have been developed as a novel alternative method with the advantage of more efficient utilization of space and less consumption of materials. This method has been applied to projects located in China and other countries, e.g., Petronas Twin Towers at Kuala Lumpur City Centre in Malaysia (Thornton et al., 1997), Fujian Industrial Bank (Shenzhen) Tower (Xu et al., 2003; Fu et al., 2003), Customer Service and Technical Support Center Building of China PING AN INSURANCE Group (Li, 2006), and Tianhe Fangyuan Hotel in Guangzhou (Zhu et al., 2012). Fig.2 illustrates a typical overlapping column transfer assembly constructed using reinforced concrete (RC) to implement the vertical retraction of the building. The assembly components generally consist of upper and lower columns, upper and lower beams with connected slabs, and a transfer block. A critical parameter is the overlapping ratio \( e/h \), where \( e \) denotes the offset distance of the two columns and \( h \) means the height of the transfer block.

Efforts have been made to understand the structural behavior of overlapping column transfer assemblies constructed using RC. As presented in Fig.2, the assembly is featured with a pair of large axial forces, \( N_1 \) and \( N_2 \), in the misaligned columns C1 and C3, respectively. The two axial forces, on the one hand,
primarily generated an overturning moment and induced compression, \( N_3 \), in beam B1 and tension, \( N_4 \), in beam B2. On the other hand, large axial forces generally caused significant vertical shear across the transfer block C2. Tests verified that the assemblies could reasonably transfer vertical forces in misaligned columns (Xu et al., 2003; Quan et al., 2006). In addition, numerical simulations (Gu and Fu, 2003) and shaking table tests using scaled models (Lu et al., 2013) revealed that buildings using overlapping column transfer assemblies had even stiffness along a vertical direction. However, undesired soft storey under earthquakes was not expected during the tests.

Generally, the shear bearing capacity of the transfer block C2 is a critical issue in the structural design of whole assemblies (Xu et al., 2003; Li, 2006; Zhu et al., 2012). An appropriate design procedure for transfer blocks is currently unavailable in design codes, e.g., ACI318-11 (2011) or GB50010-2010 (2014). Xu et al. (2003) suggested that the design method for transfer blocks was similar to that for shear walls under shear loads because a shear failure of transfer blocks could occur. Thus, the specified shear bearing capacity of RC transfer blocks, \( V_c \), is formulated per Chinese code (GB50010-2010, 2014) as:

\[
V_c \leq 0.15 f_c b h
\]

where \( f_c \) is the specified axially compressive strength of normal concrete. Meanwhile, \( b \) and \( h \) denote the width and height, respectively, of the transfer block cross-sectional area that suffered large vertical shear.

In instances involving limited dimensions, the shear bearing capacity of a transfer block constructed using RC has been unable to meet the Chinese design requirement. As an alternative, a steel truss has been selected by the industry to encase in the transfer block to improve its shear bearing capacity. This structural system is new and of practical significance for general engineering scenarios. However, knowledge of the structural performance of steel-encased overlapping column transfer assemblies is not found in the codes and literature. In this regard, tests were conducted on two specimens scaled to prototype dimensions following a verified numerical model and parametric studies. Special attention was paid to understand the fundamental mechanical behavior of this type of structural assembly.

2. Experimental Program

2.1 Prototype Assembly

The experimental program was motivated by the design of high-rise buildings located in Nanchang, China. The prototype structures were twin 50-story reinforced concrete buildings, each built with an inner core tube. The façade of each building retracted vertically on the 32nd floor, resulting in an offset of the relevant perimeter columns, as illustrated in Figs.3.a and 3.b. The overlapping column transfer assembly was adopted to achieve the transfer of vertical forces in the misaligned columns. However, preliminary calculations concluded that an assembly solely using RC could not meet the design requirement of the transfer block expressed in Eq. (1). Afterwards, a detailed finite element analysis supported this conclusion. Consequently, a modified assembly encased with steel was put forward to improve the shear bearing capacity of the transfer block, as presented in Fig.3.c.

2.2 Determination of Model for Testing

The purpose of this subsection is to obtain a reasonable 1:1 scaled model for easy testing in laboratory conditions. Meanwhile, the mechanical behavior of the test model should be in conformance with that of the prototype assembly. Fig.4. presents six candidate models with consideration of various loads, lengths of columns and beams, and support conditions. In each model, the assembly components were assumed to behave elastically, and the loads were applied at the moment inflection point at approximately mid-height of the upper column. The applied loads, i.e., the axial force \( N \), and shear force \( V \), were extracted from computational results of the prototype structure. The load combination of "1.2 x dead loads + 1.4 x live loads" was used per Chinese code GB50009-2012 (2012). As an example, Fig.5. illustrates the internal force diagrams of the columns and beams of model f. Table 1. compares the internal forces in critical cross-sections of columns and beams of the candidate models with those of the prototype structure. Results indicated that the stress distribution of transfer blocks in an individual candidate model was similar throughout and, thus, its stress distribution is not presented here for conciseness.
However, model $f$ was thought to be superior to other models because its internal forces in the critical cross-sections were closer to those of the prototype structure among the six candidate models. Thus, model $f$ was adopted as a benchmark of scaled specimen for testing.

2.3 Test Setup

Two 1:6 scaled specimens were manufactured for testing in conformance with the candidate model $f$. Fig.6. and Table 2. present the dimensions and reinforcements of specimens A1 and A2. They were constructed using RC, and the only difference between them was that specimen A2 was encased with a steel truss. The truss was manufactured using steel plates with a thickness of 5 mm. Both specimens had an overlapping ratio of 0.56. The concrete was cast simultaneously and had a compressive strength (prism 100 x 100 x 300 mm) of 53.1 MPa, an elastic modulus of $3.74 \times 10^4$ MPa, and a tensile strength of 3.89 MPa.

Three reinforcements were used: 1) the deformed 10 mm diameter reinforcing steel bars for longitudinal reinforcements, 2) the round 10 mm diameter reinforcing steel bars for stirrups, and 3) steel plates. They each had yield strengths of 423 MPa, 718 MPa, and 299 MPa, ultimate strengths of 592 MPa, 796 MPa, and 440 MPa, and elastic moduli of $1.82 \times 10^5$ MPa, 1.75 x
MPa and $1.82 \times 10^5$ MPa, respectively.

Fig. 7 presents the test setup. For each specimen, the ends of the upper beam L1 and lower beam L2 were fastened on the reaction frame using bolts and pre-embedded steel plates at the beam-ends. The foot of the lower column C3 was affixed with the base because they were monolithically cast. Vertical and horizontal loads, $F$ and $H$ with a constant ratio of $F:H = 4.2:1$, were applied at points A and B on the head of the upper column C1 (see Fig. 6.a) and increased proportionally until specimen failure. This ratio was determined according to the computational internal forces at the moment inflection point of the prototype's upper column C1 under rare earthquakes. In testing, the specimens were prevented from lateral instability by using lateral steel supports.

Fig. 6.a also presents two displacement meters installed at points A and C, corresponding to the point for applying vertical loads and the point on the bottom of transfer block C2, respectively. The strains were measured by affixed strain gauges on the surfaces of the reinforcing steel bars and on the steel truss in the midsection region of the transfer block, as presented in Fig. 8. In addition, strain gauges were also placed on the upper and lower columns to monitor their stress states but are not presented here for conciseness.

### 2.4 Test Results

Shear failure was observed in the transfer block C2 of Specimen A1 with a vertical peak load of 1322 kN. Cracking originated in the bottom region of beams L1

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| Region of midsection in grey background | Strain gauges were affixed in longitudinal bars |
|----------------------------------------|---------------------------------------------|
| I Section                              | A B C D                                     |
| II Section                             | E F G H                                    |
| III Section                            | Strain gauges were affixed in stirrups      |
| C2                                     |                                            |

| Strain gauges arranged in reinforcing steel bars in transfer blocks C2 of Specimens A1 and A2 |
|---------------------------------------------------------------------------------------------|
| (a)                                                                                         |

| Strain gauges and strain gauge rosettes affixed on surface of steel truss of Specimen A2 |
|--------------------------------------------------------------------------------------------|
| (b)                                                                                         |

| Dimensions and Reinforcements of Specimens A1 and A2 |
|-------------------------------------------------------|
| Dimension/Component | C1 | C2 | C3 | L1 and L2 |
|----------------------|----|----|----|-----------|
| Cross-sectional Width (mm) | 135| 135| 135| 135 |
| Cross-sectional Height (mm) | 350| 730| 350| 167 |
| Length (mm) | 436| 675| 634| 1160 |
| Longitudinal Reinforcement | 14D10 | 28D10 | 14D10 | 4D10 |
| Stirrup | D10@80 | D10@80 | D10@80 | D10@120 |

Note: D10 means diameter of 10 mm.
and L2 adjacent to the transfer block C2 at a loading level of about 200 kN. The lengths and number of cracks along the beam length in the lower beam L2 increased gradually. This was because the beam L2 was under tension as indicated in Fig.5. In the midsection region of the transfer block C2, fine cracks angled from 45 to 60 degrees with respect to the horizontal direction when loads grew to about 550 kN. These fine cracks developed in length and width and then formed critical cracks almost in a vertical direction. Finally, the specimen failed as shown in Figs.9.a and 9.b. Unlike Specimen A1, Specimen A2 failed due to eccentric compression of the upper column C1 as presented in Figs.9.c and 9.d, with a vertical peak load of 1422 kN. Cracks in the steel-encased transfer block C2 were also observed from the time when 500 kN loads were applied, but these loads did not cause specimen failure. Evidently, the steel-encased transfer block of Specimen A2 escaped failure because of its improved shear bearing capacity compared to that of Specimen A1. Thus, the weak part of Specimen A2 was moved from the transfer block C2 to the upper column C1.

Fig.10. shows the relation between vertical force F and vertical displacement at point C. The original stiffnesses of the two specimens were similar to each other and then significantly decreased when loads increased beyond approximately 500 kN. The ultimate displacements were approximately 6 mm for both specimens.

Table 3. Comparison of Numerical Results with Test Data

| Load and Displacement | Simulation | Test | Error   |
|------------------------|------------|------|---------|
| Peak load (kN) Specimen A1 | 1330       | 1322 | +0.75%  |
| Specimen A2           | 1490       | 1422 | +4.9%   |
| Ultimate displacement at point C (mm) Specimen A1 | 5.66       | 5.48 | +3.3%   |
| Specimen A2           | 4.48       | 6.08 | -26%    |
| Cracking load for L2 (kN) Specimen A1 | 275        | 300  | -       |
| Specimen A2           | 280        | 250  | -       |
| Cracking load for C2 (kN) Specimen A1 | 480        | 550  | -       |
| Specimen A2           | 540        | 500  | -       |

Note: Errors for cracking loads were not provided because visual observation of the cracks in tests was approximate.

3. Numerical Model and Verification

A three-dimensional finite element method-based model was developed in this section with the purpose of providing a tool for parametric studies. The model was built using the commercial software ABAQUS (ABAQUS, 2009). The mechanical behavior of concrete was described using a damage plasticity model. This model required definition of the material's uniaxial constitutive relationships and yield criterion. Fig.12. presents the models for concrete under uniaxial compression (GB50010, 2014) and under tension with consideration of stress softening, respectively. The yield criterion proposed by Lubliner et al. (1989) was adopted because it matched experimental data quite well and was suggested by the software. The constitutive equation based on a nonassociated flow rule was used with the plastic potential function in a form of Drucker-Prager type. The material properties of the reinforcing steel bars and steel plates were simulated by using a bilinear elastic-plastic model and Von Mises yield criterion. The Poisson's ratios were set to 0.2, 0.3 and 0.3 for concrete, reinforcing steel bars and steel plates.

Three element types were adopted: 1) the hexahedral eight-node, linear, and reduced-integration element (C3D8R) for concrete, 2) the two-node truss element (T3D2) for reinforcing steel bars, and 3) the four-node doubly curved shell element with reduced integration (S4R) for steel plates. The slip behaviors between them were not considered. The commonly used mesh size of
50 mm x 50 mm x 50 mm was adopted for concrete. The bars and steel plates were embedded in concrete. Fig.13 presents the meshed solid elements for concrete and steel plates.

The undesired zero energy modes (hourglass modes) due to reduced integration were appropriately controlled by defined reasonable hourglass stiffness. The boundary condition in the numerical model was set to coincide with that in the tests. The force control mode was applied according to the testing protocol. The equations were solved by Newton's iterative procedure.

Table 3 compares the numerical results with test data. Fig.10 presents the individual relation between vertical force and vertical displacement obtained from the tests and simulation. The vertical peak loads were well simulated with errors of less than 5%. However, the calculated displacement at point C of Specimen A2 was less than the test result by 26%. A possible reason was that the bond behavior between concrete and steel plates was not considered in the simulation. This increased the specimen stiffness, resulting in a relatively small displacement compared to test observation.

In addition, Fig.14 compares the computational cracking distributions of transfer blocks in Specimens A1 and A2 before specimen failure with those of test observation. The computational cracking was represented using maximum principal plastic strain in concrete, which was suggested by software ABAQUS based on the research conducted by Lubliner et al. (1989). Results indicated that the principal plastic strains primarily concentrated in the vertical region of the midsection in the two specimens. In Specimen A2, the cracking also extended to the foot of the upper column C1. The computational cracking distributions agreed well with test observations.

In summary, the developed numerical model can well describe the mechanical behavior of the assemblies and, thus, was used for further parametric studies.

4. Parametric Studies

The effects of critical parameters on the load bearing capacity of the steel-encased transfer block C2 are presented in this subsection. The critical parameters are considered to be the sharp optimization of the steel truss, cross-sectional steel plate ratio, and overlapping ratio. For the specimens used in the parametric studies, the concrete and arrangement of the reinforcing steel bars were identical to Specimens A1 and A2. The dimension and steel plates were varied for specific specimens. In addition, the following assumptions were adopted for each specimen:

1. The failure of the assembly was limited to its transfer block. The columns and beams escaped failure.

2. Horizontal loads $H$ applied on top of the upper column had almost no influence on the shear bearing capacity of the transfer block and were not considered.

Assumption 1 was achieved by numerically strengthening the columns and beams connected to the assemblies. Assumption 2 was confirmed by two comparative studies presented in Fig.15. The numerical relations of the vertical loads and vertical displacements for Specimens A1 and A2 with and without horizontal loads were obtained. All specimens failed due to shear in the transfer blocks. Results revealed that horizontal loads had almost no influence on the shear bearing capacities of the transfer blocks. The horizontal loads $H$ caused limited variation in the horizontal force and moment in the vertical potential failure section in the transfer block. However, for a sufficiently reinforced section to transfer shear, its shear resistance was not sensitive to the magnitude of the variation of the horizontal force and moment in this study case (Lin et al., 2016; Mattock et al., 1975).

This understanding was also consistent with previous experimental studies in which the horizontal loads were ignored (Xu et al., 2003; Quan et al., 2006).

Additionally, failure due to eccentric compression was observed in the test in the upper column of Specimen A2 with a peak load of 1422 kN. However, Fig.15 indicates that the peak load of Specimen A2 increased to about 1790 kN when shear failure occurred in its transfer block instead of eccentric compression failure in the upper column. That is, when both Specimens A1 and A2 failed due to shear in their transfer blocks, an increase of the peak load by 35%, i.e., from 1322 kN to 1790 kN, could be expected.
4.1 Sharp Optimization of the Steel Truss

Relatively high stresses in the steel truss were observed in the midsection region of the transfer block of Specimen A2 in tests. In light of this observation, a sharp optimization of the steel truss in Specimen A2 was performed. The ideas were to change the diagonal struts X1 and X2 to horizontal struts. This resulted in an increase of 50% in the cross-sectional area of the steel plates across the transfer block midsection, while the volumetric steel plate ratio remained unchanged. Fig.16. presents the optimized steel truss sharpness and the relations of vertical force and vertical displacement before and after optimization for comparison. The peak load of the optimized specimen achieved about 1880 kN and increased by approximately 7.5% compared to 1790 kN of the original specimen.

4.2 Effect of Cross-sectional Steel Plate Ratio

A group of five specimens with different cross-sectional steel plate ratios and optimized steel trusses as presented in Subsection 4.1 were designed to calculate their peak loads. The cross-sectional steel plate ratios were limited to about 10% for the sake of economic benefit (JGJ138-2001, 2002). Consequently, the cross-sectional steel plate ratios of 1.86%, 2.79%, 3.73%, 7.46% and 10.2% were set for the five specimens, respectively. The cross-sectional steel plate ratio of 3.73% was the case of the optimized specimen. Fig.17. presents the peak vertical forces of the five specimens which improved from 1610 kN to 2230 kN with the increase in steel plate ratios from 1.85% to 10.2%. This indicates that a steel truss with a relatively large cross-sectional area helps to resist the shear across the region of the transfer blocks' midsection.

4.3 Effect of Overlapping Ratio

A group of six specimens was designed with different overlapping ratios $e/h$, optimized sharpness of the steel truss and a cross-sectional steel plate ratio of 3.73%. The overlapping ratios were set to 0.2, 0.4, 0.52, 0.56, 0.6 and 0.8. Fig.18. presents the calculated normalized peak vertical forces, $F_{\text{max}}/f_{bh}$, with respect to the overlapping ratios. A similar specimen group without steel truss was also designed with their data given in the figure. Results indicated that the peak loads significantly increased with the decrease in overlapping ratios for both specimen groups. The increase in magnitudes of peak loads was about 3.5 times for steel-encased assemblies and about 5 times for assemblies without steel truss. This trend was consistent with the generally accepted shear theory of reinforced concrete beams, i.e., relatively large moment-to-shear ratios lead to a decrease in shear bearing capacities (Wight and MacGregor, 2011).

In addition, two best fitting curves for the prediction of peak loads with respect to overlapping ratios are presented in Fig.18. for design applications.
5. Recommendations for Structural Design

The structural performance of a transfer block in a specific assembly should be better than that of the connected columns and beams. The reason lies in the function and failure risk of transfer blocks. Evidently, the function of a transfer block is similar to that of a column-beam joint in a frame structure (Lu et al., 2015). The failure of a transfer block could result in the destruction of all the structural components connected to it.

A possible nonconformity should be eliminated in the numerical treatment between the global model and local model of transfer blocks. Transfer blocks were generally assumed to behave elastically in the modeling of the whole structure. This is because a type of robust and efficient modeling for the description of transfer blocks rather than an elastic body is currently unavailable. This assumption, however, needs to be verified in an appropriate local model extracted from a global one. This paper used a detailed finite element method-based analysis which is recommended to consistently ensure the elastic behavior of transfer blocks.

The test results obtained from the 1/6-scaled specimens can be different from those of original members due to the size effect of materials, especially of quasi-brittle material like concrete. Generally, the size effect is responsible for the strength decrease of concrete and components when the specimens become large in size. Consequently, an overestimation of the shear bearing capacity of the prototype assembly is expected when based solely on the test results.

6. Conclusions

Mechanical behavior of steel-encased overlapping column transfer assemblies was experimentally and numerically investigated. Two comparative specimens were first tested. A finite element method-based model was then built and verified using test data. The model was further used as a basis for intensive parametric studies. Finally, design recommendations were proposed concerning elastic behavior of steel-encased transfer blocks and size effect. Based on these studies, the following conclusions could be drawn:

1) Compared to the assemblies solely constructed using RC, significant improvement of the load bearing capacities was confirmed for steel-encased assemblies. Under conditions of shear failure in transfer blocks with an overlapping ratio of 0.56, the load bearing capacity of the steel-encased assembly with a cross-sectional steel plate ratio of 3.73% increased by 35% compared to that of the RC assembly.

2) Optimum truss with a relatively large steel area across the shear plane was recommended to improve the assembly's load bearing capacity. An optimized steel truss sharp is presented in Fig.16. In addition, the load bearing capacities improved with the increase in cross-sectional steel plate ratios varying from 1.86% to 10.2%, and decreased in overlapping ratios in a range of 0.2 to 0.8.

A practice-oriented expression for the prediction of load bearing capacities of steel-encased assemblies was beyond the scope of this study. This can be achieved in future research when more test data and numerical analyses are available.

Acknowledgements

The authors are grateful for the financial support from the National Natural Science Foundation of China under grant no. 51578399.

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