Study on the Shear Capacity of an SRC-RC Transfer Column

Wei Huang*, 1, 2, Jiang Qian3 and Zhi Zhou4

1 Lecturer, Hubei Key Laboratory of Theory and Application of Advanced Materials Mechanics (Wuhan University of Technology), China
2 Lecturer, Department of Mechanics and Engineering Structure, Wuhan University of Technology, China
3 Professor, State Key Laboratory of Disaster Reduction in Civil Engineering, Tongji University, China
4 Doctoral Candidate, State Key Laboratory of Disaster Reduction in Civil Engineering, Tongji University, China

Abstract
The truncation of steel sections in Steel Reinforced Concrete-Reinforced Concrete (SRC-RC) transfer columns may lead to a distortion in the internal force transmission. A refined finite element model of SRC-RC transfer columns was developed in this study taking into consideration the effects of different confinement around concrete. According to the internal force and deformation characteristics of a transfer column, the shear capacity of the SRC-RC transfer column was investigated, and the results compared well with existing experimental results. Extensive parametric studies were conducted and crosschecked with the current code. It is found that the current specification may not always ensure the shear failure capacity since it is intrinsically affected by both the internal force and deformation characteristics of the transfer column. A modified formula has been proposed, by which more rational design of structural members may be expected, brittle failure can be avoided and ductility will be improved.

Keywords: SRC-RC transition column; transmission distortion; finite element model; shear capacity

1. Introduction
A Steel Reinforced Concrete-Reinforced Concrete (SRC-RC) transfer column behaves as the connection between the upper and lower members by extending the steel of the SRC or steel column to the adjacent members. Suzuki et al. (1999) investigated the influence on the mechanical behavior of transfer columns by extending the height of steel. Low frequency cyclic tests of 3 transfer columns and 1 RC column were conducted to compare their behaviors. Konno et al. (1998) tested 13 specimens of transfer members under cyclic load to obtain the seismic performance of transfer columns. Cyclic tests of 7 SRC-RC transfer columns were performed by Kimura et al. (1998), to evaluate the effects of the enhanced reinforcement and extending the height of steel on the seismic performance and mechanical behavior of the transfer columns. Xue et al. (2010) and Wu et al. (2012) tested 21 SRC-RC transfer columns under cyclic load. The results found that the specimens were mostly dominated by shear failure, and the flexural failure and bond failure were mainly observed in specimens with greater extending height of steel. The influence of different structural measures on the seismic performance of transfer columns was studied through the cyclic tests of 8 SRC-RC transfer columns by Zhao and Shao (2010).

In the Chinese Code for Design of Concrete Structures GB 50010-2010, only a few structural measures are presented. However, calculation methods of transfer columns, such as formulas for flexural capacity and shear capacity, are not provided. The shear capacity formula for transfer columns in the Japanese Code, which is derived from the ultimate flexural performance of transfer columns was studied through the cyclic tests of 8 SRC-RC transfer columns by Zhao and Shao (2010). Thus, it is difficult for engineers to evaluate the performance of transfer columns in structural design. Though several experimental studies have been conducted, no systematic results have been achieved because the number of tests and experimental conditions are limited. To generally investigate the behavior of SRC-RC transfer columns, the constitutive model of concrete, which considers the effect of different confinements is used according to the principle of effective additive confined stress. Then, the finite element model for an SRC-RC transfer column is established in ABAQUS. Then, the security and rationality of the shear capacity calculation for SRC-RC transfer columns are explored.
2. Finite Element Modeling

Finite element analysis (FEA) was performed by ABAQUS to develop reliable models that can simulate the behavior of SRC-RC transfer columns, based on experimental results.

2.1 Modeling Approach

The SRC-RC transfer column studied in this paper is composed of two parts, the SRC part and the RC part. The SRC part includes four components, the steel section, longitudinal reinforcement bars, transverse reinforcement bars and concrete. Furthermore, the RC part is divided into three zones, which are highly confined concrete, partially confined concrete and unconfined concrete zones with different states of confinement, as shown in Fig.1.

Fig.1. Modeling of SRC-RC Transfer Column

The highly confined concrete is taken from the web of steel to the mid-width of each flange outstand. The partially confined concrete is taken from the mid-width of each flange outstand to the centerline of the longitudinal reinforcement. Finally, the unconfined concrete is the remaining external zone, as shown in Fig.2.

Fig.2. Confinement Zones in Concrete

2.2 Finite Element Type and Modeling of Interfaces

The steel and concrete were modeled by an 8-node solid element with reduced integration (C3D8R), and the reinforcements were modeled by 3-D linear truss elements (T3D2). The interface elements (using the *Contact Pair option) available within the ABAQUS element library (SIMULA, 2011) were used to model the interaction between the steel-section and concrete.

The master surface in the contact property is defined as the concrete surface surrounding the steel. When the two surfaces remain in contact, the slave surface can be displaced in relation to the master surface based on the coefficient of friction between the two surfaces, which is taken as 0.25 (Ellobody et al., 2011). Moreover, the reinforcements were embedded into the concrete using the *Embedded Constraint option in ABAQUS, assuming a perfect bond.

2.3 Material Modeling of Steel Section and Reinforcement Bars

The stress-strain curves for structural steel and the reinforcement bars, provided by the Chinese Code for Design of Concrete Structures GB 50010-2010 (2010), were adopted in this study with values of yield stress \( f_{ys} \) and ultimate stress \( f_{us} \) measured in the tests. The material behavior provided by ABAQUS (using the *PLASTIC option) allows a nonlinear stress-strain curve to be used. The first part of the nonlinear curve represents the elastic behavior up to the proportional limit stress with a Young’s modulus of 200 GPa and Poisson’s ratio equal to 0.3.

2.4 Material Modeling of Concrete

In this case, the concrete strength is considerably improved because of confinement by the steel and reinforcements. Thus, a confined concrete model needs to be developed, based on the experimental results. The model proposed by Popovics (1973) and Collins et al. (1993) was adopted to simulate the concrete behavior in compression. The compressive stress \( f_c \) and strain \( \epsilon_c \) relation of the concrete is defined as follows:

\[
\sigma_c = \frac{f_c \lambda (\epsilon_c / \epsilon_0)}{1 - \left(\frac{\epsilon_c}{\epsilon_0}\right)^d} 
\]

\[
\lambda = \frac{E_c}{f_c} \left(\frac{\epsilon_c}{\epsilon_0} \right) 
\]

\[
d = \begin{cases} 
1, & \epsilon_c / \epsilon_0 \leq 1 \\
0.67 + \frac{f_c}{62}, & 1 < \epsilon_c / \epsilon_0 < 1 \\
1, & \epsilon_c / \epsilon_0 \geq 1 
\end{cases}
\]

where \( f_c \) is the ultimate compressive strength of concrete with or without confinement, \( \lambda \) is the curve-fitting factor, \( E_c \) is the initial tangent modulus, \( \epsilon_0 \) is the strain when \( \sigma_c \) reaches \( f_c \), and \( d \) is the factor which controls the slope of the stress-strain curve. Fig.3. shows the equivalent uniaxial stress–strain curves for both unconfined and confined concrete.

Fig.3. Compression Stress-strain Model of Concrete
The initial Young’s modulus of concrete is reasonably calculated using Eq. (4) given by the ACI Specification (2011). The Poisson’s ratio of concrete is taken as 0.2.

\[ E_c = 4700 \sqrt{f_c} \]  

(4)

1) Unconfined Concrete

\[ f_c = f_{c0} \]  

(5)

where \( f_{c0} \) is the ultimate compressive strength of unconfined concrete and \( \varepsilon_{c0} \) is the strain when \( \sigma_c \) reaches \( f_{c0} \) (taken as 0.002 in this case).

2) Partially Confined Concrete

The compressive strength \( f_c \) and the corresponding strain \( \varepsilon_c \), of confined concrete can be determined by Eqs. (5) and (6), respectively, proposed by Mander et al. (1988) and improved by Denavit et al. (2011).

\[ f_c = K f_{c0} + \varepsilon_{c0} (1 + 5(K - 1)) \]  

(6)

\[ K = 1 + Af_c (0.1 + \frac{0.9}{1 + Bf_c}) \]  

(7)

3) Highly Confined Concrete

High confinement of concrete is provided by the steel and reinforcements. The lateral confining pressure \( f_{cl} \) imposed by the flange of the steel is given by Huang et al. (2016):

\[ f_{cl} = K_f f_{c0} \]  

(12)

\[ f_{cl} = \frac{f_{cl2} + f_{cl1}}{2} f_{c0} \]  

(10)

\[ r = f_{cl2} / f_{cl1} \leq f_{cl2} \]  

(11)

where \( f_{cl} \) is the equivalent lateral confining pressure, \( f_{cl1} \) and \( f_{cl2} \) are the lateral confining pressures imposed by the reinforcement bars with different directions, respectively, as given by Mander et al. (1988).

The ultimate compressive strain \( \varepsilon_{cu} \) of confined concrete is given by Paulay et al. (2009).

\[ \varepsilon_{cu} = 0.004 + 1.4 \rho_f f_{y} \varepsilon_{min} / f_c \]  

(15)

where \( \varepsilon_{min} \) is the strain when the tensile stress of the steel bar reaches the maximum, and \( \varepsilon_c \) is the transverse reinforcement ratio. The ultimate compressive strain of the confined concrete varies from 0.012 to 0.05, which is 4 to 16 times that of the unconfined concrete.

Concrete was modeled using the damaged plasticity model implemented in the ABAQUS standard and explicit material library. Under uniaxial tension, the stress–strain response follows a linear elastic relationship until reaching the value of the failure stress. The tensile failure stress was assumed to be 0.1 times the compressive strength of concrete. The softening stress–strain response, past the maximum tensile stress, was represented by a linear line defined by the fracture energy and crack bandwidth, as shown in Fig.4. The fracture energy \( G_f \) in N/mm (energy required to open a unit area of crack) was taken as Eq. (16) as recommended by the CEB Specification (1996):

\[ G_f = \alpha(0.1f_y)^{0.7} \]  

(16)

where \( f_y \) is the compressive strength in MPa; the coefficient \( \alpha \) is concerned with the maximum diameter of concrete aggregate, and can be taken as \( D_{max} = 8 \) mm, \( a = 0.025; D_{max} = 16 \) mm, \( a = 0.03; D_{max} = 32 \) mm, \( a = 0.058 \).

3. Validation of the Finite Element Model

Based on the simulation method above, numerical analyses of 9 transfer columns are performed to study the nonlinear mechanical behavior of transfer columns (Suzuki et al. 1994, Kimura et al. 1998, Xue et al. 2010), as shown in Table 1.

The load-displacement curves of the FEM analyses are compared with those of the tests in Fig.5. The corresponding load eigenvalues and ductility coefficients are listed in Table 2. The yield point is determined by using the energy equivalent method, while the ultimate lateral displacement is considered to be the corresponding displacement to the shear capacity when the load decreases to 85% of the capacity in the load-displacement curve. It can be found that the results of the finite element analysis agree with those of the tests. The FEM reflects the pinching effect and
the damage conditions of the members under cyclic load, as well as the decline after the maximum load. An obvious difference between the numerical result and test result is observed in specimens under continuous beam loading, as shown in Figs.5.(1) and (2), because the next load step is conducted before the stress of the member becomes stable. It is caused by the principle of one-time loading. However, the overall trend of the results is similar.

Table 1. Dimensions and Test Parameters of the Specimens

| Reference          | Test No. | Dimension (mm) | Steel section | Extending height coefficient of steel | Stirrup configuration | Stirrup enhancement at truncated place of steel | Longitudinal reinforcement |
|--------------------|----------|----------------|---------------|--------------------------------------|-----------------------|-----------------------------------------------|---------------------------|
| Suzuki et al. 1994 | S3-30    | 400×400 1200   | H204×200×12×12| 0.3                                  | Φ10@120               | --                                            | 8Φ19                      |
|                    | S3-60    | 400×400 1200   | H204×200×12×12| 0.6                                  | Φ10@120               | --                                            | 8Φ19                      |
| SRC4-4-JM          | S4-2-JM  | 220×160 1000   | H100×68×7.4×4.5| 0.4                                  | Φ6.5@48               | --                                            | 4Φ16                      |
| Kimura et al. 1998 | SRC6-2   | 220×160 1000   | H140×80×9.1×5.5| 0.4                                  | Φ6.5@96 Φ6.5@48      | 4Φ16                                        |
| SRC8-2             | SRC8-4   | 220×160 1000   | H140×80×9.1×5.5| 0.8                                  | Φ6.5@96 Φ6.5@48      | 4Φ16                                        |
| Xue et al. 2010    | SRC-2-JM | 220×160 1000   | H140×80×9.1×5.5| 0.4                                  | Φ6.5@96 Φ6.5@48      | 4Φ16                                        |

Fig.5. Comparison between Tests and Finite Element Results

4. The Shear Capacity of the Transfer Column

The change in stiffness caused by the local existence of steel results in a change of the inflection point location at approximately 0.4 times the column height that is near the RC part, thus easily generating a short-column shear failure. The steel and concrete in the SRC part mutually squeeze together along the height of the column. The concrete transfers part of the shear and axial forces to the steel. The co-working of the concrete and steel is based on the force transfer.
Table 2. Comparison between Tests and Finite Element Results

| Reference       | Test No. | Yield load/kN | Q_{T,\text{TEST}} | Q_{T,\text{FEA}} | Shear capacity/kN | V_{T,\text{TEST}}/V_{T,\text{FEA}} | Ductility coefficient μ | μ_{T,\text{TEST}}/μ_{T,\text{FEA}} |
|-----------------|----------|---------------|-------------------|-----------------|-------------------|----------------------------------|-------------------------|-------------------------|
| Suzuki et al.   | S3-30    | 345.00        | 354.00            | 0.97            | 401.00            | 394.00                          | 1.02                    | 2.14                    |
|                 | S3-60    | 325.00        | 343.50            | 0.95            | 400.00            | 384.80                          | 1.04                    | 3.32                    |
|                 |          |               |                   |                 |                   |                                  |                         |                         |
| Kimura et al.   | S4-2-JM  | 90.04         | 91.00             | 0.99            | 119.60            | 123.00                          | 0.97                    | 5.99                    |
|                 | SRC4-4-JM| 114.75        | 112.00            | 1.02            | 147.35            | 147.00                          | 1.00                    | 3.39                    |
|                 | SRC6-2   | 99.55         | 99.80             | 1.00            | 127.65            | 132.70                          | 0.96                    | 3.78                    |
|                 | SRC8-2   | 104.30        | 106.00            | 0.98            | 133.20            | 136.00                          | 0.98                    | 3.63                    |
|                 | SRC8-4   | 113.70        | 120.00            | 0.95            | 146.25            | 144.00                          | 1.02                    | 3.54                    |
| Xue et al.      | SRC-2-JM | 102.05        | 104.00            | 0.98            | 134.70            | 142.00                          | 0.95                    | 4.65                    |
|                 | SRC-2-SD | 98.70         | 96.90             | 1.02            | 130.65            | 128.00                          | 1.02                    | 3.30                    |

between them, which also results in the distortion of the internal force transmission. Fig.6. shows the ideal failure surface of a transfer column under a compound stress condition. Fig.7. presents the stress distribution of the reinforced concrete part of a transfer column. According to Figs.6.-7., the failure angle $\theta$ ($\theta \leq 45^\circ$) between the column axis and the trace of the principle tension stress can be obtained by Eqs. (17)-(18). The lower limitation of $\theta$ is determined by cot $\theta = 2$, which is provided by Zhao (1998). The minimum value of $\theta$ is 26.6$^\circ$.

\[ \tau = \sqrt{f_t^2 - \sigma_0 f_t} \]  
\[ \theta = \min \left\{ \tan^{-1} \left( \frac{h}{L - L_s} \right), \frac{1}{2} \tan^{-1} \left( \frac{2 \tau}{\sigma_0} \right) \right\} \]

where $\sigma_0 = N/(bh)$; $b$ and $h$ are the width and height of the section; $f_t$ is the compressive strength of concrete; $f_t$ is the tensile strength of concrete; and $h_s$ is the space of longitudinal reinforcements.

The modified Mohr-Coulomb failure criterion is adopted as the three black polylines presented in Fig.8. Under the combination of shear stress and normal stress, shear slip failure occurs when the Moore stress circle is tangent to the oblique line, and principle tensile failure occurs when the Moore stress circle is tangent to the vertical line.

According to the geometric relations of the Moore stress circle, the maximum shear stress of the section is determined by:

\[ \frac{A_s}{f_s} = \frac{f_t}{1.5k} \]

where the factor $k$ is taken as 1.2.

The stirrups not only provide shear capacity, but also limit the cracking of the concrete. In considering of the failure mode of the transfer column, the shear capacity of the stirrups $V_s$ is reduced and is more conservative than the code.

\[ V_s = 0.6 A_s f_s / s \]

where $A_s$ is the area of stirrups; $f_s$ is the yield strength of reinforcement; $s$ is the space of stirrups.

Considering the shear capacity of both stirrups and concrete, the shear capacity of a transfer column can be determined by:

\[ V = V_c + V_s \]

When the shear span ratio is greater, the flexural behavior becomes dominant. Meanwhile, the flexural capacity of the steel can be fully developed, thus delaying the flexural yielding at the bottom of the SRC part and improving its shear capacity. The shear capacity of a transfer column under flexural failure is determined by the flexural capacity of the ends of...
the column. The ultimate flexural capacity of the RC and SRC parts can be obtained by Eqs. (23) and (24), respectively, according to the Japanese code (Feng et al., 1998). Then, the shear capacity of a transfer column under flexural failure can be calculated by Eq. (25).

\[ M_{RC} = 0.8 A_f f_y h + 0.5 N_h (1 - N / (b h f_c)) \]  \hspace{1cm} (23)

\[ M_{SRC} = 0.8 A_f f_y h + 0.5 N_h (1 - N / (b h f_c)) + W f_y \]  \hspace{1cm} (24)

\[ V_2 = (M_{RC} + M_{SRC}) / L \]  \hspace{1cm} (25)

where \( A_f \) is the cross-section area of tensile longitudinal reinforcement, \( h \) is the height of the section, \( h_0 \) is the space of longitudinal reinforcement, \( W \) is the elastic resistance moment of steel, \( f_y \) is the yield strength of steel, and \( f_c \) is the yield strength of a reinforcement bar.

Eqs. (22) and (25) show the shear capacity of a transfer column under different failure modes. The shear capacity can be calculated as:

\[ V = \min(V_1, V_2) \]  \hspace{1cm} (26)

Based on Eq. (26) above, the test results by Suzuki et al. (1994) and Kimura et al. (1998) are compared in Table 3, which indicates that the values obtained by Eq. (26) agree with the test results. Though some values are conservative, it is reasonable to increase the strength of a transfer column. The shear capacity obtained by the principle tension method differs greatly from that of the flexural shear capacity method, which is caused by the failure characteristics of a transfer column. Hence, measures can be taken to prevent brittle failure and improve ductility.

The Chinese Code for Design of Concrete Structures (2010) provides the formula of shear capacity for RC members under eccentric compressive load as follows:

\[ V_{code} = \frac{1.75}{\lambda + 1} f_y b h_0 + f_y v \frac{A}{s} \frac{A}{s} h_0 + 0.07N \]  \hspace{1cm} (27)

where the factor \( \lambda \) is the shear span ratio and \( h_0 \) is the effective height of the section.

The stirrups not only provide shear capacity, but also limit the cracking of the concrete, preventing shear failure. To improve safety, the shear capacity is reduced by the stirrups. The shear capacity is modified as follows:

\[ V_{mod} = \frac{1.75}{\lambda + 1} f_y b h_0 + 0.6 f_y v \frac{A}{s} \frac{A}{s} h_0 + 0.07N \]  \hspace{1cm} (28)

The shear capacities of a transfer column obtained by the code and the proposed modified formula are shown in Table 4. It can be found that the shear capacity obtained by the code is safe when the stirrups have a large space, but is not safe when the space of the stirrups gets smaller. For a transfer column, the local existence of steel causes a change in the inflection point near the RC part, thus easily generating a short-column shear failure. Moreover, when the location of the inflection point changes, the shear span ratio of the RC part tends to decrease. As is commonly known, the
bigger the shear span ratio, the lower the shear capacity is. Therefore, the calculation of the shear capacity for a transfer column is much safer when the inflection point is simply considered to be at the center of the column according to the code.

To validate the rationality and reliability for the proposed modified formula for the shear capacity, the above finite element method is adopted to establish the models of specimens as in Suzuki et al. (1994). The dimensions of section (B × D) are 400 mm × 400 mm, and the other parameters of the specimens are presented in Table 5.

The values calculated by the code and the modified formula are compared with the ultimate shear capacity of the tests, as shown in Table 6. It can be found that the shear capacity calculated by the code is close to the test results. However when the space of stirrups gets smaller, the shear capacity tends to be greater and unsafe. Meanwhile, the values obtained by the modified formula are approximately 0.7-0.9 times that of the test values, providing enough redundancy and safety.

6. Conclusions
In an SRC-RC transfer column, the sudden change in stiffness caused by the local variation in the steel section will result in moment distribution changes at the ends of the column. Thus, in consequence, the inflection point of the column may move toward the RC part. Based on the mechanical behavior of SRC-RC transfer columns, the shear capacity formula under different failure modes has been studied. The calculated values agree with the test results. It is found that the shear capacity formula provided by the current code may not always ensure the shear failure capacity while the proposed modified formula, which reduces the shear capacity provided by stirrups, will lead to a more rational design of structural members by preventing brittle failure and improving ductility.
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