Evaluation Method of Shear Capacity of Steel Reinforced Concrete Beam Considering Shear-span Ratio and Support Condition

Yuki NAKATA Ken WATANABE Toshiya TADOKORO Masaru OKAMOTO
Concrete Structures Laboratory, Structures Technology Division

Manabu IKEDA
Steel and Hybrid Structures Laboratory, Structures Technology Division

A number of calculation equations for the design shear capacity of steel reinforced concrete (SRC) members with simple support are given in the Design Standards for Railway Structures and Commentary (Concrete Standard). In addition, experiments for Railway Structures and Commentary (Concrete Structures) [1] (Hybrid Standard). These equations have been formulated for SRC beams with simple support considering of the effect of the shear span effective depth ratio (a/d) on the shear capacity and the knowledge of reinforced concrete (RC) deep beams. However, there are some equations that are applicable to a certain member because the scope of application for these equations is unclear. In addition, support conditions for transverse beams on railway viaducts are different from simple supports because both transverse beam ends are fixed. In this study, the scope of application for existing equations was clarified, and an equation to calculate the shear capacity of SRC beams under asymmetric moment distribution was proposed.

Keywords: steel reinforced concrete, shear capacity, shear span effective depth ratio, support condition, steel-frame ratio

1. Introduction

A number of calculation equations for the design shear capacity of steel reinforced concrete (SRC) members with simple support are given in the Design Standards for Railway Structures and Commentary (Steel-Concrete Hybrid Structures). However, there are some equations that are applicable to a certain member because the scope of application for these equations is unclear. In addition, support conditions for transverse beams on railway viaducts are different from simple supports because both transverse beam ends are fixed. In this study, the scope of application for existing equations was clarified, and an equation to calculate the shear capacity of SRC beams under asymmetric moment distribution was proposed.

$$V_{sd} = f_{yld} \cdot \frac{z_w \cdot w_d}{t_h} \quad (\gamma_h = 1.15)$$

where, $f_{yld} = (1000/d)^{0.20} \leq 1.5, \beta_d = (100p_c)^{0.20} \leq 1.5, p_c = A_d/(b_w \cdot d), f_{cd} = 0.20 \cdot f'_{cd}^{0.15} (N/mm^2), V_{cdi}$ design shear capacity of linear members without shear reinforcing steel (N), $V_{cdi}$: design shear capacity of stirrups, $V_{cd}$: design shear capacity of steel frame, $d$: effective height (mm), $A_d$: area of tension reinforcement (mm$^2$), $b_w$: web width (mm), $f'_{cd}$: design compressive strength of concrete (N/mm$^2$), $A_s$: total area of stirrups placed in $s$ (mm$^2$), $f_{s,yd}$: design yield strength of stirrups, $\theta$: angle between stirrups and member axis, $z$: spacing of stirrups, $z_w$: distance from the location of compressive stress resultant to the centroid of tension steel, $f_{yld}$: design shear yield strength of steel (N/mm$^2$), $t_h$: web height of steel frame, $z_w$: web thickness of steel frame.

Equation (1) is a calculation equation which is assumed to be applied to various SRC members, for example, various support and load conditions, and members for which the a/d is uncertain. Thus, the shear capacity obtained using (1) must be estimated to be smaller than the actual shear capacity under various conditions.

Figure 1 shows the comparison of $V_{yld}$ and experimental results. The experimental results were obtained through experiments on SRC beams with simple support [3] and SRC beams with both ends fixed, in chapter four. Also, the $\gamma_h$ of $V_{yld}$ and $V_{sd}$ were set at 1.1 in reference to Standard Specifications for Hybrid Structures in JSCE [4]. From Fig 1, $V_{yld}$ at a stirrup ratio $p_s = A_d/(b_w \cdot s)$ was 0.48% was larger than the experimental result.

Figure 2 shows the strain distributions of stirrups on specimens at $p_s = 0.48 \% \text{ at } V_{exp}$ (refer to 4.1). The strains of stirrups have not reached the yield strain. Because the yield of stirrups is presupposed in $V_{yld}$, the application of $V_{yld}$ is undesirable. It is thought that $V_{yld}$ can be estimated to be smaller than the actual shear capacity if the upper limit of $p_s$ used for $V_{yld}$ is 0.22%, of which all specimens reached the yield strains (Fig 1), though the upper limit
changes according to the specifications of members [5].

3. Evaluation method of the shear capacity of SRC beams with simple support considering the effect of \( a/d \)

Equation (2), (3) are calculation equations for SRC beams with simple support considering the effect of \( a/d \) given in the Hybrid Standard.

\[
V_{\text{eff}} = V_{\text{sd}} + V_{\text{sd}}^\prime + \alpha V_{\text{sd}} \tag{2}
\]

\[
V_{\text{sd}} = \beta_a a/d \cdot f_{cd1}^{1/2} \cdot \beta_p b_s \cdot d / \gamma_b \quad (\gamma_b = 1.3)
\]

\[
\beta_a = 0.20 \cdot (0.75 + 1.4 \alpha a/d), \quad \alpha \geq 2.5
\]

\[
= 0.76 \cdot (a/d)^{1.10}, \quad 0.5 \leq a/d \leq 2.5
\]

\[
\alpha = 2.7 + 0.16 k - 0.68(a/d),
\]

\[
1.0 \leq a/d \leq 3.5, \quad 2.0 \leq k \leq 7.0, \quad 0.6 \leq \alpha \leq 2.5 \tag{3}
\]

\[
V_{\text{sd}}^\prime = V_{\text{dd}} + V_{\text{dd}} \cdot h < 2.0 \quad \text{(simple beam)}
\]

\[
V_{\text{dd}} = \beta_d \cdot \beta_p \cdot b_s \cdot f_{cd2} \cdot d / \gamma_b \quad (\gamma_b = 1.3)
\]

where, \( \beta_a = 5/[(1+a/d^2)], f_{cd1} = 0.19 f_{cd1}^{1/2}, a/d: \) shear span effective height ratio, \( k: \) steel-frame ratio \((=100 \cdot A_s (b_s \cdot d))\) (%), \( A_s: \) area of steel frame, \( l: \) span of beam, \( h: \) section height of beam. \( V_{\text{dd}} \) is the sum of \( V_{\text{dd}}, \) which is the design shear capacity of a deep beam, and \( V_{\text{sd}} \) [1]. Moreover, \( \beta_d \) is 1.0 in this study.

The applicable scope of (3) is \( h < 2.0 \) (simple beam), and it nearly equals to \( a/d = 1.0 \) in the case that the distance from the support front end to the loading point \( a \) is \( 2.0 \). So that, it follows that both (2) and (3) are applicable in the region of a small \( a/d \).

By the way, \( V_{\text{sd}} \) \((0.5 \leq a/d \leq 2.5)\) of (2) and \( V_{\text{dd}} \) of (3) interfaced with the equation based on the experiments of footings [6] and the design shear capacity of deep beams. These equations, however, were unified and modified into the equation of shear compression capacity \( V_{\text{dd}}, \) which considered the effect of stirrups in the Concrete Standard [2].

\[
V_{\text{dd}} = \beta_d \cdot \beta_p \cdot b_s \cdot f_{cd2} \cdot b_s \cdot d / \gamma_b \quad a/d < 2.0 \quad (\gamma_b = 1.2.0) \tag{4}
\]

where, \( \beta_a = 4.2 \cdot (100 p_{w}^{1/2} \cdot (a/d - 0.75) / \gamma_b^{1/2} \) (when \( p_{w} < 0.002), \beta_a \) is taken as 0), \( \beta_p = (1+(100 p_{w}^{1/2}) / p_{w}: \) shear reinforcement ratio (when \( p_{w} < 0.002), \beta_p \) is taken as 0)

Therefore, \( V_{\text{dd}} \) shall be applicable at \( a/d < 2.0, \) and shear capacity \( V_{\text{uf}} \) of SRC beam with simple support shall be obtained using (5).

\[
V_{\text{uf}} = V_{\text{sd}} + V_{\text{sd}} \cdot 2.0 \leq a/d \leq 3.5 \tag{5}
\]

\[
= V_{\text{dd}} + \alpha \cdot V_{\text{sd}}, \quad a/d < 2.0
\]

\[
V_{\text{sd}} = 0.20 \cdot (0.75 + 1.4 \alpha a/d) \cdot f_{cd1}^{1/2} \cdot \beta_p b_s \cdot d / \gamma_b \quad (\gamma_b = 1.3)
\]

The \( \gamma_b \) of \( V_{\text{sd}} \) and \( V_{\text{uf}} \) were 1.1 [4]. Figure 3 shows the comparison of \( V_{\text{uf}}, \) which is the maximum value of the shear force \( V_{\text{uf}} \) obtained experiments on SRC beams with simple support [3]. It was confirmed that \( V_{\text{uf}} \) could estimate the experimental results \( V_{\text{uf}} \). Also, (1) shall be used when \( a/d \) is over 3.5 or unknown.

![](Fig_1.png)

**Fig. 1 Validation of accuracy of \( V_{\text{uf}} \)**

**Fig. 2 Stirrup strain distribution \( (p_{w}=0.48\%)\)**

3. Evaluation method of the shear capacity of SRC beams with simple support considering the effect of \( a/d \)

![Image](Fig_3.png)

**Fig. 3 Validation of accuracy of \( V_{\text{uf}} \)**

4. Investigation on the shear capacity of SRC beams with both ends fixed

4.1 Summary of experimental results

Table 1 shows the list of specimens of SRC beams with both ends fixed under antisymmetric moment distribution [7]. It should be noted that stirrups on specimens with \( p_{w}=0.48\% \) did not yield (Fig. 2), shear forces contributed by steel frames, stirrups and concrete interact with each other [7] and \( V_{\text{exp}} \) increases if the width of the flange decreases (Fig. 4). \( V_{\text{exp}} \) is the shear force at the point where stiffness changes just after the web or flange of the steel frame yields. It is considered that the flexural stiffness before the flexural yield point is commonly applied if a shear failure member is modeled as a beam element.

In this chapter, experimental results were reproduced using the 3D finite element method (FEM), and shear mechanisms were analytically investigated after validity of the analysis model was verified.
4.2 Analysis outline

3D nonlinear analyses were conducted with DIANA (ver.9.4.4). Figure 5 shows an example of the analysis model. The whole span and a half of the cross-section width of the specimen was discretized. Adopted elements are shown in Fig.5, and the stress transfer between the steel frame and the concrete were expressed by setting a series of interface elements. The parabolic model and the tension softening model proposed by Hordijk [8] were utilized as the concrete constitutive model for compression and tension. The fracture energy for demonstrating the softening curve was calculated according to the Concrete Standard and a study proposed by Nakamura and Higai [9].

Elements near loading and supporting points in two stubs at both ends of the test span were assumed to be a perfect elastic body having an elastic modulus. The stress-strain relationship of the rebar was expressed as bi-linear. The rotating crack model was used for the expression of cracked concrete element.

4.3 Reproduction analyses of experimental results

Figure 6 shows the comparisons of shear force- relative displacement relationships of the analyses and experiments. Analytical results, corresponding to the case where the adhesional properties between the steel frame and the concrete were perfect (Perfect bond) and did not exist (No bond), were shown. Initial stiffness of No bond corresponded to the experiments. After that, stiffness was changed with the occurrence of diagonal cracks and horizontal cracks along the longitudinal rebars and the flanges of the steel frame in the experiments, and the shear forces at points where stiffness changed in the Perfect bond were larger than in the experiments. On the other hand, stiffness after diagonal cracks occurred and $V_{exp}$ in the experiments tended to be between Perfect bond and No bond. Though the adhesional properties between steel frame and the concrete in the experiments were unknown, the analytical model was considered valid because the experimental results fell between those obtained from the analysis on the assumption of Perfect bond and those assuming No bond.

### Table 1 List of specimens

| Specimen | $b_w$ (mm) | $a/d$  | $f_c$ (N/mm²) | Longitudinal rebar | Stirrup | Steel frame | $V_{exp}$ (kN) |
|----------|------------|--------|----------------|-------------------|---------|-------------|----------------|
| SRC1     | 300        | 1.0    | 25.6           | D29 (381)         | 970     | 244 × 175 × 7 × 11 | 509            |
| SRC2     | 300        | 1.5    | 24.5           | D25 (225)         | 968     | 250 × 200 × 9 × 14 | 629            |
| SRC3     | 300        | 1.5    | 27.4           | D28 (256)         | 941     | 250 × 200 × 9 × 14 | 463            |
| SRC4     | 300        | 1.5    | 28.1           | D10 (100)         | 0.48    | 379         | 532            |
| SRC5     | 400        | 1.0    | 34.4           | D25 (225)         | 968     | 250 × 200 × 9 × 14 | 747            |
| SRC6     | 400        | 1.5    | 32.6           | D25 (225)         | 968     | 250 × 200 × 9 × 14 | 747            |
| SRC7     | 400        | 1.5    | 29.0           | D25 (225)         | 941     | 250 × 200 × 9 × 14 | 664            |
| SRC8     | 400        | 1.5    | 66.4           | D25 (225)         | 941     | 250 × 200 × 9 × 14 | 920            |
| SRC9     | 400        | 2.5    | 36.5           | D29 (286)         | 941     | 250 × 200 × 9 × 14 | 590            |
| SRC10    | 400        | 2.5    | 34.9           | D29 (286)         | 941     | 250 × 200 × 9 × 14 | 493            |
| SRC11    | 400        | 1.5    | 33.9           | D25 (225)         | 972     | 125 × 200 × 9 × 14 | 446            |
| SRC12    | 400        | 1.0    | 33.0           | D25 (225)         | 972     | 125 × 200 × 9 × 14 | 556            |
| SRC13    | 400        | 1.5    | 35.2           | D25 (225)         | 993     | 250 × 200 × 9 × 14 | 463            |

*1 SRC1-13 is buildup steel, others are rolled material, *2 Height of steel frame × width of flange × thickness of web of steel frame × thickness of flange (mm), *3 Cut flange of steel frame (rolled material) with 250 × 250 × 9 × 14mm

**Fig. 4** Relationship of shear force and relative displacement

**Fig. 5** An example of analysis model ($a/d$=1.0)
### 4.4 Investigation of shear mechanisms

#### 4.4.1 Outline of parameter analyses

Analyses with parameters such as thickness of the web of the steel frame $t_w$, stirrup ratio $p_w$, width of flange and the adhesional properties between the steel frame and the concrete, were conducted using the model shown in 4.3 to investigate the shear mechanisms and the effect of reinforcement with steel.

Table.2 shows case specific analyses. The longitudinal rebar was elastic and the elastic modulus was $2.0 \times 10^5$ N/mm$^2$. The yield strength and elastic modulus of the stirrups were 380 N/mm$^2$ and $2.0 \times 10^5$ N/mm$^2$, respectively. The compressive strength of concrete $f'_c$ was 27 N/mm$^2$. The tension strength and elastic modulus of the concrete were calculated using the Concrete Standard. The fracture energies of compression and tension were 50 N/mm and 0.10 N/mm, respectively. The yield strength and elastic modulus of the steel frame were 300 N/mm$^2$ and $2.0 \times 10^5$ N/mm$^2$, respectively. The flanges of the steel frame were elastic.

#### 4.4.2 Effect of the stirrup ratio $p_w$

Figure 7 shows stirrup strain distribution at the maximum value of shear force $V_{\text{ana}}$ obtained from the analyses. Stirrup strain fell with the increase in $p_w$, and did not reach yield strain at $p_w = 0.23\%$. This confirmed that the minimum principal stress stood out in the compression zone of both ends of the test span. Thus, it is thought that stirrups become less likely to yield because the damage of the concrete precedes the yield of the stirrups with the increase in $p_w$. In the other cases, stirrup strain also fell with the increase in $p_w$ though the distribution shapes of stirrup strain were different among the specimens.

Figure 8 shows examples of the relationship between $p_w$ and $V_{\text{ana}}$. In each case, stirrups become less likely to yield at $p_w = 0.09\%$. Figure 9 shows an example of the relationship between $p_w$ and $V_{\text{ana}}$. In each case, stirrups become less likely to yield at $p_w = 0.09\%$.

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**Table 2 Analyses cases**

| CASE | Base specimens | Property of interface | Width of flange (mm) | $a/d$ | Stirrup ratio $p_w$ (%) | Thickness of web of steel frame $t_w$ (mm) |
|------|----------------|----------------------|---------------------|------|------------------------|------------------------------------------|
| 1    | SRC1 ~ 4      | No bond              | 175                 | 1.0  | 1.5  2.0  0.00  0.10  0.20  0.25  0.30  0.35  0.40  0.45  0.50 | 3  6  9  12  15                        |
| 2    | Perfect bond   |                      |                     | 1.0  | 1.5  2.0  0.00  0.10  0.20                  |                                          |
| 3    | No bond        |                      | 250                 | 1.0  | 1.5  | 0.00  0.05  0.09  0.19  0.23                  | 3  6  9                                 |
| 4    | Perfect bond   |                      |                     | 1.0  | 1.5  | 0.00  0.05  0.09  0.19  0.23                  |                                          |
| 5    | No bond        |                      | 113                 | 1.0  | 0.00  0.05  0.09  0.19  0.23                  | 3  6  9                                 |
| 6    | Perfect bond   |                      |                     | 1.0  | 0.00  0.05  0.09  0.19  0.23                  |                                          |

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**Fig. 6 Relationship of shear force and relative displacement (Comparison of analyses and experiments)**

**Fig. 7 Stirrup strain distribution (CASE 4)**

**Fig. 8 The effect of $p_w$ on $V_{\text{ana}}$ (CASE 3,4) ($a/d=1.0$)**

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yield at relatively small values of $p_w=0.19$-0.50 \%. It is found that the rate of increase in $V_{\text{uana}}$ falls against as $p_w$ increases.

4.4.3 Effect of the width of the flange

Figure 9 shows the relationships between $p_w$ and the ratio of $V_{\text{uana}}$ for CASE 5 to 3 and CASE 6 to 4. Regarding the analytical conditions, the only difference between CASE 5 and 3, and CASE 6 and 4, is flange width. In common with each comparison, $V_{\text{uana}}$ ratios had a tendency to be over 1.0 in case $p_w$ or $t_w$ is small. $V_{\text{uana}}$ ratios decreased $p_w$ or $t_w$ increased.

When flange width was small, the flexural yield of both ends of the steel frame preceded the shear yield of center with the increase in $p_w$ or $t_w$. As a result, the region of the shear yield of the steel frame decreases. Thus, the ratios of $V_{\text{uana}}$ decrease with the increase in $p_w$ or $t_w$ because the shear force contributed by the steel frame decreases in addition to the increase of the shear force contributed by steel. This tendency, which was caused by the flexural yield of both ends of the steel frame, was confirmed in the experiments (comparison between SRC9 and SRC10).

Figure 10 shows an example of the distributions of minimum principal stress at $V_{\text{uana}}$. This was shown in eight elements (eight layers) divided width ways. The amplitude of the minimum principal stress and the distributional width became large in all layers as the width of the flanges decreased. In particular, as the flange got smaller, a significant amount of the load was transferred and $V_{\text{uana}}$ increased because the amplitude of the minimum principal stress and the distributional width of the layers (1-6 layers) positioned outside the flanges were large.

5. Proposed calculation equation for the shear capacity of SRC beams with both ends fixed in consideration of the effect of $a/d$

A calculation equation for the shear capacity of SRC beams with both ends fixed for a small $a/d$ was proposed on the basis of the aforementioned experiments and analyses. The proposed equation was expressed by adding the effect of the steel frame to the calculation equation of the shear capacity of RC beams with both ends fixed $V_{\text{yd4}}$ [10].

$$V_{\text{yd4}} = V_{\text{cd4}} + V_{\text{atd}}, \quad 1 \leq a/d \leq 2.0 \quad (6)$$

$$V_{\text{cd4}} = (1.0 - 0.75 + 4.0(a/d)/f_{\text{cd}}p_w f_{\text{yd}} b_w d / \gamma_h \quad (\gamma_h = 1.3)$$

$$V_{\text{atd}} = p_\gamma \cdot p_\gamma' \cdot \gamma_b \cdot \cot \theta \cdot \exp \{0.44 \times (a/d) - 0.35 p_\gamma + 0.58 \} \leq 1.0$$

Figures 11 shows the comparisons of the values, which is after subtracting $V_{\text{cd4}}$ ($\gamma_h = 1.0$) in consideration of the upper limit of $p_w$ and $V_{\text{atd}}$ ($\gamma_h = 1.0$) from $V_{\text{exp}}$ (SRC1-7, 11-13), and $V_{\text{atd}}$. A steel ratio $k$ % was selected as a comprehensive parameter, which expresses the interaction of the effect of the reinforcement by the steel frame and the effect of the width of the flanges. It was found that $(V_{\text{exp}}-V_{\text{cd4}})/V_{\text{cd4}}$ correlated with $k$. Therefore, shear capacity contributed by material other than the steel $V_{\text{cd4}}$ shall be obtained from (7), in which $V_{\text{cd4}}$ is multiplied by (1-0.08k). (1-0.08k) was obtained by linear regression in Fig.11, and when $k=0$, (1-0.08k) shall be 0.

$$V_{\text{cd4}}=1-(1-0.08k) \cdot V_{\text{cd4}} \quad (7)$$

where, $3.0 \leq k(\%) \leq 5.1$.

The equation to calculate the shear capacity of SRC beams with both ends fixed is shown as (8).

$$V_{\text{yd4}}=V_{\text{cd4}}+V_{\text{atd}}+V_{\text{et}}, \quad 1 \leq a/d < 2.0 \quad (8)$$

where, $V_{\text{et}}$: when $p_w>0.22$ %, $p_w$ is taken as 0.22 %.

Figure 12 shows the comparisons between $V_{\text{yd4}}$ and $V_{\text{exp}}$. It was confirmed that $V_{\text{yd4}}$ could be used to accurately estimate the results obtained by experimental means, and all
experimental results could be safely estimated by considering $\gamma_b$. Also, (1) shall be used when $a/d \geq 2.0$.

6. Conclusions

(1) The scope of application of $a/d$ in the case of SRC beams with simple support, was clarified for existing equations while matching the calculation method of shear capacities to the one in the Concrete Standards. An equation to calculate shear capacity of SRC beams with simple support in consideration of the effect of $a/d$ was proposed.

(2) For SRC beams with both ends fixed, stirrups became less likely to yield because the damage of the concrete preceded the yield of the stirrups with the increase in $p_w$. So, it was found that the shear capacity of stirrups had an upper limit.

(3) The shear capacity increased with the decrease in the width of the flanges because the amplitude of the minimum principal stress and the distributional width of the concrete outside the flanges were large.

(4) An equation to calculate the shear capacity of SRC beams with both ends fixed in consideration of the effect of the $a/d$ was proposed on the basis of the experiments and analyses.

The contents of this paper will be described in the Design Standards for Railway Structures (Steel-Concrete Hybrid Structures).

Acknowledgement

The contents of this paper have been discussed in the committee on Design Standards for Railway Structures (Steel-Concrete Hybrid Structures). The authors would like to express their appreciation for contributions from committee members, in particular, Prof. Dr. Tamon Ueda (Hokkaido University) : chairman, Prof. Dr. Akinori Nakajima (Utsunomiya University) : secretary-general.

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Authors

Yuki NAKATA
Researcher, Concrete Structures Laboratory, Structures Technology Division
Research Areas: Concrete Structures

Ken WATANABE, Ph.D.
Assistant Senior Researcher, Concrete Structures Laboratory, Structures Technology Division
Research Areas: Concrete Structures

Toshiya TADOKORO, Dr. Eng.
Senior Researcher, Concrete Structures Laboratory, Structures Technology Division
Research Areas: Concrete Structures

Masaru OKAMOTO, Dr. Eng.
Laboratory Head, Concrete Structures Laboratory, Structures Technology Division
Research Areas: Concrete Structures

Manabu IKEDA, Dr. Eng.
Laboratory Head, Steel and Hybrid Structures Laboratory, Structures Technology Division
Research Areas: Hybrid Structures, Steel Structures