Evaluation Formula on Punching Shear Load-Carrying Capacity of RC Slabs under the Running-Load

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The reinforced concrete slabs of steel road bridges directly support the wheel loads of large vehicles. The maximum load-carrying capacity of the slabs is usually evaluated by a punching shear load-carrying capacity. With the objective of clarifying the punching shear load-carrying capacity of the slabs under running loads, the authors conducted tests under running loads on the three types of test specimens with different compressive strength of concrete, quantity of reinforcements, and effective depth. As a result, the authors proposed a punching shear dynamic model in the ultimate limit state and an equation for calculating the punching shear load-carrying capacity based on the maximum load-carrying capacity and the strains in reinforcements, and evaluated the agreement between the experimental values and the theoretical values calculated from the equation.

1. INTRODUCTION

The reinforced concrete (RC) slabs of steel road bridges directly support the wheel loads of large vehicles. The maximum load-carrying capacity of the slabs is usually evaluated by the punching shear load-carrying capacity. As regards the punching shear load-carrying capacity of RC slabs under running loads, Matsui et al. conducted fatigue tests on RC slab test specimens under wheel loads and proposed a punching shear dynamic model and an equation for calculating the punching shear load-carrying capacity of fatigue-damaged RC slabs.5, 6 The authors conducted tests on two types of steel road bridge RC slab test specimens under running loads and proposed a punching shear dynamic model and an equation for calculating the punching shear load-carrying capacity of RC slabs.7 In addition, the authors proposed an equation for calculating the shear strength of concrete and applied an equation for calculating the punching shear load-carrying capacity.8 However the proposed model and equation were applicable near the fracture load. Accordingly, the authors proposed a punching shear dynamic model and an equation for calculating the punching shear load-carrying capacity of RC slabs under static loads in the ultimate limit state near the yield load, tensile strength, and fracture load, and evaluated the appropriateness of the proposed equation.9

In this study, in order to theoretically evaluate the punching shear load-carrying capacity of steel road bridge RC slabs under running loads, the authors conducted the tests on three types of RC slab test specimens that were designed in accordance with the Specifications for Highway Bridges II (hereinafter referred to as the Specifications II)10 under running loads. Based on the maximum load-carrying capacity and the load-strain relationship obtained from the tests, the authors proposed a punching shear dynamic model and an equation for calculating the punching shear load-carrying capacity of RC slabs in the ultimate limit state near the yield load, tensile strength and fracture load, and evaluated the agreement between the experimental values and the theoretical values calculated from the equation to contribute toward establishing the rational design method of steel road bridge RC slabs.

2. MATERIALS AND SIZE OF TEST SPECIMENS AND TEST METHOD

2.1 Materials of test specimens

Ordinary Portland cement and coarse aggregate with maximum size of 20 mm were used for the concrete of test specimens. Reinforcements SD295A D10, D13 and SD295A D10 were used for Types I, II and III test specimens, respectively. Table 1 lists the mechanical properties of the materials used for this test.
2.2 Size of test specimens and arrangement of reinforcements

Assuming that the planned traffic volume of heavy vehicles per day in one direction was 500 vehicles or less for Types I and II test specimens and 2,000 vehicles or more for Type III, the thickness of slabs was determined in accordance with the Specifications II, and a half-scale model was used for the tests.

Type I test specimen was isotropic 147 cm in total length and 120 cm in span, with reinforcements arranged on both tension and compression sides. On the tension side, reinforcements D10 were placed every 10 cm and 12 cm in the transverse and longitudinal direction, respectively. The effective depth was 9 cm and 8 cm. On the compression side, reinforcements in quantity half those on the tension side were arranged. Type II test specimen was the same as Type I test specimen in terms of the spacing of reinforcements and effective depth but 1.7 times larger than Type I test specimen in the quantity of reinforcements that reinforcements D13 were used. Type I and II test specimens are hereinafter described as I-R- and II-R-. Type III test specimen was the same as Types I in terms of total length and span. Reinforcements for Type III were arranged on both tension and compression sides. On the tension side, reinforcements D10 were placed every 10 cm in the transverse and longitudinal directions. On the compression side, reinforcements in quantity half those on the tension side were arranged. The effective depth was 10.5 cm and 9.5 cm. Type III is hereinafter described as III-R-. RC slab test specimens were square plates simply supported on four sides. The RC slab test specimens were square plates with all edges simply supported. The dimensions of the plates and the arrangement of reinforcements for Types I and II are shown in Fig. 1 (1) and those for Type III in Fig. 1 (2)

![Dimensions of test specimens (unit in mm)](image)

2.3 Tests on RC slabs under running loads

The tests on RC slabs under running loads were conducted by moving the load from the center of the span, where the load-bearing capacity was significantly reduced under the running loads, through both support points to the center of the span: the wheel was first set at the center of the span; the load was exerted on the specimen at the center; the load was moved in the longitudinal direction; the load was traveled between both support points; and the load was brought to the center of the span. The traveling speed was 13 seconds in one travel of 2.4 m, i.e. 0.18 m/sec. The incremental load was applied: the magnitude of the load was increased in increments of 5.0 kN per travel. Strains in main reinforcements and distribution bars were measured at the center of slabs. The travel range of running load and the measuring point of strain are shown in Fig. 1.
3. TEST RESULTS AND DISCUSSIONS

3.1 Experimental punching shear load-carrying capacity

Maximum experimental load-carrying capacity and failure mode of RC slabs obtained from the tests are listed in Table 2.

The averages of maximum load-carrying capacities obtained from the tests on RC slabs under running loads were 126.0 kN for Type I and 149.8 kN for Type II test specimens. Because reinforcements D13 were used for Type II test specimens, the punching shear load-carrying capacity of Type II increased by a factor of 1.19 times as compared with Type I. The failure mode was a punching shear failure at the center of the slab during an increase in the load. The averages of maximum load-carrying capacities of Type III test specimens III-R-1 and -2 and III-R-3 and -4 were 170.2 kN and 172.0 kN, respectively. The difference in the experimental load-carrying capacities of Type III test specimens was due to the difference in the compressive strength of concrete.

The failure mode of the test specimens subjected to the tests under running loads was a punching shear failure at the center of the slab during an increase in the load. As regards the failure condition that affects a punching shear dynamic model, a shear fracture developed at an angle of 45 degrees from the wheel contact surface (25 cm × 4 cm) to the slab bottom in the transverse direction and the concrete fractured at the bottom. In the longitudinal direction, a crack penetrated the test specimen.

3.2 Relationship between load and strain

The relationship between the load acting on the slab test specimen and the strain induced in the tension reinforcement at the center of the slab is shown in Fig. 2. The strain induced in the tension reinforcement in the transverse direction in Fig. 2 is shown in Fig. 2 1), and the strain in the distribution reinforcement, i.e. the tension reinforcement in the longitudinal direction, is shown in Fig. 2 2). The average of strains in the main and distribution reinforcements is shown in Fig. 2 3). The yield strain in the tension reinforcement, calculated from the mechanical properties in Table 1, are $1,825 \times 10^{-3}$ and $1,855 \times 10^{-3}$ for Type I and Type II test specimens, respectively. The yield strain for Type III test specimens III-R-1 and -2 and III-R-3 and -4 are $1,850 \times 10^{-3}$ and $1,840 \times 10^{-3}$, respectively. From the relationship between these yield strains and loads, a punching shear dynamic model and the equation for calculating the load-carrying capacity near the yield strength were proposed. The strain in the reinforcements at the tensile strength were $2,550 \times 10^{-3}$, $2,530 \times 10^{-3}$, $2,560 \times 10^{-3}$ and $2,580 \times 10^{-3}$ for Type I, Type II, Type III test specimens III-R-1 and -2, and Type III test specimens III-R-3 and -4, respectively. From the relationship between these strains and loads, a punching shear dynamic model and an equation for calculating the punching shear load-carrying capacity when the load reached the tensile strength were analyzed.

(1) Strain in the main reinforcements

For Type I test specimen, as shown in Fig. 2 (1 1), the tensile strain in the main reinforcements arranged in the transverse direction reached the yield at a load of about 90 kN and the tensile strength at a load of about 105 kN, and increased linearly with an increase in the load. For Type II test specimen, because its quantity of reinforcements was 1.7 times more than Type I test specimen, the yield load was increased to 110 kN and the strain increased linearly near the fracture load. For Type III test specimen, as shown in Fig. 2 (2 1), because its slab thickness was larger than Type I by 2 cm and the distribution reinforcements were arranged in the same quantity as the main reinforcements, the yield load was increased to 115 kN, the strain increased linearly near the tensile strength and significantly from a load of about 150 kN.

(2) Strain in the distribution reinforcements

For both Type I and II test specimens, as shown in Fig. 2 (1 2), the tensile strain in the distribution reinforcements arranged in the longitudinal direction increased almost linearly to near the tensile strength and significantly with an increase in the load. For Type III test specimen, as shown in Fig. 2 (2 2), the strain increased linearly near the load of 160 kN and significantly with an increase in the load. Because a crack penetrated the slab under the running loads and the slab behaved like a beam, the strain in the main reinforcements increased but the increase in strain in the distribution reinforcements from a load of 120 kN to 160 KN became small.
(3) Average of the strains in the main and distribution reinforcements

To analyze the theoretical load-carrying capacity equation, the vicinity of the yield load, tensile strength and fracture load in the ultimate limit state had to be established based on the strains in the main and distribution reinforcements. To simplify the complex analysis, the vicinity of the yield load, tensile strength and fracture load in the ultimate limit state are assumed from the average of the strains in the main and distribution reinforcements and the appropriateness of the theoretical load-carrying capacity equation was evaluated. Average of the strains in the main and distribution reinforcements is shown in Fig. 2 (3).

The loads at which the yield strain and tensile strength strain were reached were calculated from the loads that induced the strains at the yield strength and tensile strength obtained from the mechanical properties in Table 1 by the interpolation method based on the average of the strains in the main and distribution reinforcements shown in Fig. 2 (1) 3). The loads at which the yield strain was reached were 94.0 kN and 95.5 kN for Type I test specimens I-R-1 and I-R-2; 114.7 kN and 114.6 kN for Type II test specimens II-R-1 and II-R-2; 115.2 kN and 111.4 kN for Type III test specimens III-R-1 and III-R-2; and 116.0 kN and 116.7 kN for Type III test specimens III-R-3 and III-R-4, respectively. The loads at which the tensile strength strain was reached were 107.5 kN and 108.9 kN for Type I test specimens I-R-1 and I-R-2; 132.1 kN and 130.9 kN for Type II test specimens II-R-1 and II-R-2; 138.2 kN and 132.6 kN for Type III test specimens III-R-1 and III-R-2; and 141.7 kN and 142.2 kN for Type III test specimens III-R-3 and III-R-4, respectively. The agreement between the loads at which the yield strength and tensile strength were reached, which were obtained from the tests, and the loads obtained from the proposed equation for calculating the punching shear load-carrying capacity was evaluated. Table 3 lists the loads at which the yield and tensile strength strains were reached.
4. THEORETICAL PUNCHING SHEAR LOAD-CARRYING CAPACITY EQUATION

Matsui et al. proposed a punching shear dynamic model and an equation for calculating the punching shear load-carrying capacity of fatigue-damaged RC slabs. The proposed equation was derived by the sum of the punching shear load-carrying capacity affected by the shear strength from the neutral axis to the top of RC slab and the punching shear load-carrying capacity that takes into account the size effect produced by the dowel effect in the cover concrete over the tension reinforcements. The authors conducted tests on steel road bridge RC slab test specimens under running loads and proposed a punching shear dynamic model and an equation for calculating the punching shear load-carrying capacity of RC slabs based on the failure condition to modify Matsui's equation. Further, the authors conducted a single shear test on square column test specimens with a compressive strength of 20–80 N/mm², proposed the equation for calculating the shear strength of concrete, evaluated the consistency between experimental and theoretical values by applying the equation for calculating the shear strength of concrete to an equation for calculating the punching shear load-carrying capacity. In this study, with the objective of clarifying the theoretical punching shear load-carrying capacity under running loads, the punching shear load-carrying capacity in the ultimate limit state are evaluated in the following three steps: (1) the maximum punching shear load-carrying capacity when the yield strength of reinforcements is applied, i.e., near the yield strength, is evaluated, (2) the punching shear load-carrying capacity in the vicinity where the stress in the main reinforcement reaches the tensile strength, i.e., near the tensile strength, and (3) the punching shear load-carrying capacity near the fracture load.

4.1 Equation for calculating the punching shear load-carrying capacity near the yield strength

Matsui's equation was derived from a punching shear dynamic model based on the fatigue tests on steel road bridge RC slabs under wheel loads. The authors evaluated the maximum load-carrying capacity obtained from the tests under running loads that increased with traveling distance as the punching shear load-carrying capacity. However, it is desired to evaluate the design punching shear load-carrying capacity under running loads near the yield load of the tension main reinforcements. Accordingly, a punching shear dynamic model and an equation for calculating the punching shear load-carrying capacity were studied with the effect of the shear strength of concrete and the dowel effect taken into account. A punching shear dynamic model under running loads near the yield strength and an equation for calculating the punching shear load-carrying capacity were proposed by modifying a punching shear dynamic model proposed by the authors and adding a new aspect to the model.

For the calculation of the punching shear load-carrying capacity near the yield strength, a punching shear dynamic model near the load at which tension reinforcements yield was studied from the load-strain relationship of the tension reinforcement obtained from the tests under running loads.

(1) Punching shear dynamic model showing the effect of concrete shear strength and the equation for calculating load-carrying capacity

To calculate the punching shear load-carrying capacity affected by the shear strength of concrete under running loads, the size \((a)\) of the equivalent stress block in the direction of main reinforcement and the size \((a_t)\) of the equivalent stress block in the direction of distribution reinforcement are determined and the mean value \((a = (a + a_t)/2)\) of the sizes is applied. For the calculation of the size \((a, a_t)\) of the equivalent stress block in the direction of main and distribution reinforcements, the yield strength \((f_{n}, f_{n'})\) of reinforcements is used, and the quantity of reinforcements per one meter is calculated. A punching shear dynamic model is shown in Fig. 3. The punching shear load-carrying capacity affected by the shear strength of concrete is given by Eq. (1).

The equation proposed by the authors applies to the equation for calculating the shear strength of concrete, \(f_{b1}\), in Eq. (1). Okamura's equation applies to the equation for calculating the tensile strength of concrete, \(f_t\).

\[
V_{eq} = f_{n} \left\{ 2(B + 2a)a + 2(A + a_t) \right\}
\]

\[
f_{b1} = 0.688a_{b}^{1/6} \leq f_t = 80\text{N/mm}^2
\]

where

- \(A, B\): length of the sides of a loaded slab in the direction of main and distribution reinforcements, respectively, (mm), \(a, a_t\): the mean value of the size of the equivalent stress block in the direction of main reinforcement (\(a_t\)) and
the size of the equivalent stress block in the direction of distribution reinforcement \((a_r, \text{mm})\), \(f_{sw}\): shear strength of concrete, \((\text{N/mm}^2)\), \(f_{c'}\): compressive strength of concrete, \((\text{N/mm}^2)\).

Fig. 4 shows the size of the equivalent stress block, \(a_r\), in the rectangular cross section where reinforcements are arranged on both tension and compression sides. The size of the equivalent stress block is given by Eq. (2) in the case where compression reinforcements yield and Eq. (3) in the case where compression reinforcements do not yield.

1) When compression reinforcements yield

\[
a_r = (A_r \cdot f_{sw} - A_s \cdot f_{sw}) / (0.85 \cdot f_{sw} \cdot b \cdot d)
\]

2) When compression reinforcements do not yield

\[
a_r = \frac{m}{d} \left\{ 2p - p(1/2) \cdot E_s / f_{sw} \cdot \right\} \left\{ p - p(1/2) \cdot E_s / f_{sw} \cdot \right\} + p^2 \cdot 4b^2 / m \cdot d^2 \cdot e_{sw} \cdot E_s / f_{sw} \}
\]

where

\(f_{sw}\): Design compressive strength of concrete (in this study, as a result of approximation of the test strength, the concrete compressive strength \(f_{sw}\) is used) \((\text{N/mm}^2)\), \(f_{sw}\): Yield strength of reinforcements \((\text{N/mm}^2)\), \(f_{sw}\): Yield strength of compression reinforcements \((\text{N/mm}^2)\), \(e_{sw}\): ultimate strain of concrete. \(E_s\): Young modulus of tensile reinforcements \((\text{N/mm}^2)\). \(E_s\): Young modulus of compression reinforcements \((\text{N/mm}^2)\). \(A_s\): Quantity of reinforcements on tension side, \(A_s\): Quantity of reinforcements on compression side, \(d\): Effective depth (\(= d_s\)), \(d_s\): Distance from compression edge to compression reinforcement central, \(b\): Member width (=100cm)

Because compression reinforcements of Types I, II and III test specimens do not yield, the size of the equivalent stress block, \(a_r\), is given by Eq. (3), where \(a_r\) and \(a_r\) are the size of the equivalent stress block in the direction of main and distribution reinforcements, respectively.

(2) Punching shear dynamic model considering the dowel effect and the equation for calculating punching shear load-carrying capacity

As is the case of a punching shear dynamic model under static loads, the mean value of the thicknesses of cover concrete \((C' = (d_s + d')/2)\), where \(d_s\) is the thickness of cover concrete over tension main reinforcements and \(d'\) is the thickness of cover concrete over distribution reinforcements, is taken as the range in which the dowel effect is exerted in a punching shear dynamic model near the yield strength of main reinforcements under running loads. The size effect, \(C_s\), produced by the dowel effect near the yield load does not extend through the entire area of cover concrete but ranges to the mean value of the thicknesses of cover concrete, \(C_s\), multiplied by the yield strength, \(f_{sw}\), divided by the tensile strength, \(f_{sw}\), and a triangle load is exerted in the range four times larger than the mean value. Fig. 5 shows the distributed surface to which the dowel effect ranges. The punching shear load-carrying capacity is given by Eq. (4).

\[
V_{p} = f_c \cdot (4(2d + B)C_s) \quad (4)
\]

\[
f_c = 0.269f_{sw} \quad (4.4)
\]

\[
C_s = C' \cdot (f_{sw}/f_r) \quad (4.4)
\]

where

\(A_s, B\): Side length in main reinforcement and distribution reinforcement direction of loaded slab \((\text{mm})\), \(C_s\): Size effect showing impact of dowel effect \((= C_s = C' \times f_{sw})\), \(C_s\): Mean value of main reinforcement covering \((d_s)\) and covering in distribution reinforcement direction \((d')\) \((= (d_s + d')/2)\), \(d_s\): Mean value of effective depth of main reinforcements \((d_s)\) and effective depth in distribution reinforcement direction \((d_c)\) \((d_s = (H - C_s))\), \(H\): Slab thickness \((\text{mm})\), \(f_{sw}\): Compressive strength of concrete \((\text{N/mm}^2})\).
(3) Punching shear load-carrying capacity near the yield strength under running loads

Punching shear load-carrying capacity near the yield strength under running loads is given by the sum of Eq. (1) for calculating the punching shear load-carrying capacity affected by the shear strength of concrete and Eq. (4) for calculating the punching shear load-carrying capacity considering the dowel effect. Accordingly, the punching shear load-carrying capacity near the yield strength is given by Eq. (5).

\[ V_{\sigma} = f_{\sigma} \{ 2B + 2d + 2(4a + c) \} + f_{c} \{ 4(2d + B)C_{s} \} \]  
\[ f_{\sigma} = 0.688f'_{c}\text{net} \leq f'_{c} = 80\text{N/mm}^2 \]  
\[ f_{c} = 0.269f'^{\text{net}}_{c} \leq f'_{c} = 80\text{N/mm}^2 \]  
\[ C_{s} = C'_{s} \left( f_{\sigma}/f_{c} \right) \]

(5.a)
(5.b)
(5.c)

4.2 Punching shear load-carrying capacity near the tensile strength

(1) Punching shear dynamic model affected by the effect of concrete shear strength and the equation for calculating punching shear load-carrying capacity

The punching shear load-carrying capacity affected by the shear strength of concrete near the tensile strength under running loads is the same as that near the shear strength. Accordingly, an equation for calculating the punching shear load-carrying capacity affected by the shear strength of concrete is the same as Eq. (1). A punching shear dynamic model near the tensile strength is shown in Fig. 6.

Instead of applying the yield strength of reinforcements, \( f_{\sigma} \), in Equations (2) and (3) for the calculation of the size of the equivalent stress block, the size of the equivalent stress block, \( a \), near the tensile strength is calculated by applying the tensile strength of reinforcements, \( f_{t} \). The size of the equivalent stress block is given by Eq. (6) in the case where compression reinforcements yield and Eq. (7) in the case where compression reinforcements do not yield. (Equations (6) and (7))

1) When compression reinforcements yield

\[ a = (A' - A' - f_{t}d) / (0.85 - f_{\sigma}/b - d) \]  

(6)

2) When compression reinforcements do not yield

\[ a/d = m/2 \{ p - p(e_{c}\cdot E_{c}/f_{c}) + \sqrt{[p - p(e_{c}\cdot E_{c}/f_{c})]^{2} + p^{2} - 4\beta m} \} \]

\[ m = f_{c}/(0.85f_{c}) \]

(7)

where,

\( f_{t} \): Tensile strength of reinforcements (N/mm²), \( b \): Member width

(2) Punching shear dynamic model considering the dowel effect and the equation for calculating punching shear load-carrying capacity

In the case where the tensile strength of reinforcements is used for the calculation, the dowel effect ranges to the mean value of the thicknesses of cover concrete, \( C_{s} \), of the thickness of cover concrete over tension main reinforcements (\( d' \)) and the thickness of cover concrete over distribution reinforcements (\( d'' \)) multiplied by the mean value of tensile strength, \( f_{t} \), and yield strength, \( f_{\sigma} \), divided by tensile strength, \( f_{t} \), and a triangle load is exerted in the range four times larger than the dimension producing the dowel effect, \( C_{sv} \), as shown in Fig. 5. The punching shear load-carrying capacity is given by Eq. (8).

\[ V_{\sigma} = f_{c}(4(2d + B)C_{s}) \]  
\[ f_{c} = 0.269f'^{\text{net}}_{c} \leq f'_{c} = 80\text{N/mm}^2 \]  
\[ C_{s} = C'_{s} \times \left\{ (f_{\sigma}/f_{c}) \right\} \]

(8.a)
(8.b)
(3) Punching shear load-carrying capacity near the tensile strength under running loads

The punching shear load-carrying capacity near the tensile strength under running loads is given by Eq. (9), with the effect of the shear strength of concrete, \( f_{c,e} \), and the dowel effect, \( f_d \), taken into account.

\[
V_s = f_d \left\{ 2(B+2a) + 2(4a+x_0) \right\} + f_s \left\{ 4(2d+B)C_t \right\}
\]

(9)

\[
f_{d,e} = 0.68f_d^{5.41} \leq f_d = 80\text{N/mm}^2
\]

(9.a)

\[
f_s = 0.269f_s^{3.22} \leq f_s = 80\text{N/mm}^2
\]

(9.b)

\[
C_t = C_d \times \left\{ \left( \frac{f_d + f_s}{2} \right) / f_s \right\}
\]

(9.c)

4.3 Maximum load-carrying capacity near the fracture load

(1) Punching shear dynamic model affected by the effect of concrete shear strength and the equation for calculating punching shear load-carrying capacity

Under the loads near the fracture load, a crack developed in the RC slab test specimen in the directions of main and distribution reinforcements. Because a crack penetrated the slab and the slab behaved like a beam, the isotropy of the slab was lost. The effective width, \( b \), of the member used in calculating the size of equivalent stress block is given by the yield strength of tension reinforcements, \( f_{y} \), divided by the tensile strength, \( f_s \), (\( f_{y}/f_s \)) multiplied by a width of one meter (\( b = 100f_{y}/f_s \)). The quantity of reinforcements per one meter is calculated. A punching shear dynamic model is the same as shown in Fig. 6.

(2) Punching shear dynamic model considering the dowel effect and the equation for calculating punching shear load-carrying capacity

The authors proposed an equation for calculating the punching shear load-carrying capacity under running loads that was derived by modifying Matsu's equation. The width of concrete fracture due to the dowel effect was 10~12 cm, which was more than 4 times the thickness of cover concrete over reinforcements of 2.5 cm. Accordingly, considering the vicinity of fracture load, the width of stress distribution on the bottom surface of concrete affected by the dowel effect, \( C_{d,e} \), is given by the mean value, \( C_{e} \), of the thickness of cover concrete over tension main reinforcements (\( d' \)) and the thickness of cover concrete over distribution reinforcements (\( d'' \)), i.e. \( C_{d,e} = C_{e} \), and a triangle load was exerted in the range four times larger than the mean value.

(3) Punching shear load-carrying capacity near the fracture load under running loads

The punching shear load-carrying capacity near the fracture load under running loads is given by Eq. (9), with the effect of the shear strength of concrete, \( f_{c,e} \), and the dowel effect, \( f_d \), taken into account.

5. THEORETICAL LOAD-CARRYING CAPACITIES

Table 3 lists the punching shear load-carrying capacities near the yield strength, tensile strength and fracture load, which were calculated from the proposed punching shear dynamic model and the equations for calculating load-carrying capacity. In addition, Table 3 lists the loads at which the yield and tensile strength strains were reached, which were calculated from the mechanical properties in Table 1 by the interpolation method based on the average of the strains in the main and distribution reinforcements shown in Fig. 2.3

<table>
<thead>
<tr>
<th>Test specimens</th>
<th>Experimental Load of Yield strain (\times 10^4)</th>
<th>Theoretical load carrying capacity (kN)</th>
<th>Load ratio</th>
<th>Experimental load of tensile strength (kN)</th>
<th>Strain (\times 10^5)</th>
<th>Theoretical load carrying capacity (kN)</th>
<th>Load ratio</th>
<th>Experimental load of maximum load carrying capacity (kN)</th>
<th>Strain (\times 10^5)</th>
<th>Theoretical load carrying capacity (kN)</th>
<th>Load ratio</th>
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<tbody>
<tr>
<td>I-R-1(D10)</td>
<td>94.0</td>
<td>1825</td>
<td>85.7</td>
<td>1.10</td>
<td>107.5</td>
<td>2550</td>
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<td>2580</td>
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<td>1.06</td>
<td>171.5</td>
<td>171.5</td>
<td>1.06</td>
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</table>
5.1 Punching shear load-carrying capacity near the yield strength

A comparison was made between the maximum load and theoretical punching shear load-carrying capacity near the yield strength in Table 3 based on the relationship between load and strain in the directions of main and distribution reinforcements arranged in the test specimens. For Type I test specimen, the ratio of experimental to theoretical load-carrying capacity was 1.10–1.11, i.e. they were in good agreement. For Type II test specimen 1.7 times larger than Type I test specimen in the quantity of reinforcements, the ratio was 1.15 and they were in good agreement. For Type III test specimen, the ratio was 1.00–1.03. For all test specimens, the experimental values were larger than the theoretical values and they were in good agreement. The design punching shear load-carrying capacity is given by incorporating material factors of concrete and steel and member factor into Eq. (5). Accordingly, this makes the design on the safe side.

5.2 Punching shear load-carrying capacity near the tensile strength

A comparison was made between the experimental and theoretical values of punching shear load-carrying capacity in the vicinity where tension reinforcements reached tensile strength in Table 3. For Type I test specimen, the ratio of experimental to theoretical load-carrying capacity was 1.06–1.08, i.e. they were in good agreement. For Type II test specimen, the ratio was 1.07–1.08, indicating that they were in good agreement for test specimens having different quantity of reinforcements. For Type III test specimen, the ratio was 1.02–1.06, indicating that they were in good agreement.

5.3 Punching shear load-carrying capacity near the fracture load

A comparison was made between the experimental and theoretical values of punching shear load-carrying capacity near the fracture load. The ratio was 1.03–1.04, 0.99 and 1.06–1.09 for Type I, II and III test specimens, respectively. They were in good agreement.

As mentioned above, for the RC slabs designed in accordance with the Specifications II, good agreement was obtained between the experimental values and theoretical values calculated from the proposed equation for calculating the punching shear load-carrying capacity under running loads, even if the slabs had different compressive strength of concrete, quantity of reinforcements, and effective depth.

6. CONCLUSION

The authors analyzed a punching shear dynamic model and the theoretical equation for calculating the punching shear load-carrying capacity of RC slabs under running loads in the ultimate limit state near the yield strength, tensile strength and fracture load. The results of the analysis are summarized below.

(1) The failure mode of the RC slab test specimens under static loads was a punching shear failure, with a shear fracture developed at an angle of 45 degrees from the wheel contact surface.

(2) Good agreement was obtained between the experimental values and theoretical values calculated from an equation for calculating the punching shear load-carrying capacity in the elastic region under running loads, which was based on a punching shear dynamic model proposed through this study, even for the test specimens having different compressive strength of concrete, slab depth and quantity of reinforcements.

(3) Good agreement was obtained between the experimental values and theoretical values of punching shear load-carrying capacity near the tensile strength, which was calculated from the proposed equation based on a punching shear dynamic model.

(4) The authors proposed equation for calculating punching shear load-carrying capacity near the fracture load by modifying the scale effect which was applied to the dynamic model and an equation for calculating the punching shear load-carrying capacity proposed by Matsui et al. As a result, good agreement was obtained between experimental values and theoretical values and the equation seems to be useful for evaluating fatigue durability.

(5) The design punching shear load-carrying capacity is given by incorporating material factors (of concrete and steel) and member factor. Accordingly, this makes the design on the safe side.

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