- Technical Paper -

SHEAR CAPACITY PREDICTION OF REINFORCED CONCRETE MEMBERS STRENGTHENED WITH ULTRA-HIGH PERFORMANCE CONCRETE OVERLAY

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ABSTRACT

This paper presents the prediction of shear capacity of reinforced concrete (RC) strengthened with ultra-high performance concrete (UHPC) overlay based on application with modifications of current design codes. Two different methods were individually used. The first method was investigated by converting the percentage of volume steel fibres in UHPC to an equivalent longitudinal steel ratio. The second method was adopted as a sum of two components of shear contributions provided by RC member and by UHPC layer. As a result, the computed shear strength showed a promising result.

Keywords: UHPC, RC members, UHPC strengthening, prediction, shear strength

1. INTRODUCTION

Previous studies on reinforced concrete (RC) members strengthened with ultra-high performance concrete (UHPC) showed that UHPC overlay significantly enhances the structural performance [1,2], due to the excellent properties of UHPC showing strain hardening and energy absorption [3,4]. Specifically, UHPC has shown a great influence on the cracking development patterns, ultimate strength and ductility.

Although several studies on RC members strengthened with UHPC have been experimentally conducted [1,2,5], few shear models are available [6]. Noshiravani and Brühwiler [6] reported analytical models for flexural-shear resistance of composite beams. An elastic-plastic fictitious composite hinge model was used for the cracking in RC members with consideration of interaction between the two elements of RC members and UHPC; however, several analytical steps were required. For conventional RC members, shear capacity can be calculated using the current design codes such as ACI318 [7], the tensile strength of normal strength concrete (NSC) is generally negligible, whereas that of UHPC should be taken into account since UHPC exhibits high tensile strength (> 8 MPa).

To date, design provisions have not yet been available to predict the shear strength of RC members strengthened with UHPC. Methods that can predict the shear strength are therefore needed. For this purpose, use with modification of existing design models of RC and/or fibre reinforced concrete (FRC) structures would be a rational approach. The analytical models should be derived from the principle concepts of current design guidelines for RC and/or FRC structures.

This paper introduces simple methods for predicting the shear strength of RC members strengthened with UHPC overlay by application with modification of current design models. Test results of four slabs strengthened with UHPC layer and one reference RC slab conducted by Yin et al. [5] are used to validate the proposed method. A summary of the current design models [7-9] is provided. Simple assumptions for modification of current design formulations adopted in this study are described. Verification between the prediction results and the test results are presented.

2. DESCRIPTION OF TEST SPECIMENS AND RESULTS

2.1 Geometry of Specimens

Section 2 describes the test specimens and the test results of RC slabs strengthened with UHPC overlay. Full details of the specimens could be found in [5]. The reference slab specimen, RE-0, was 1600 mm long with a clear span of 1200 mm and 300 mm × 100 mm cross section. The specimens were made upside down from the state shown in Fig. 1. Before casting of UHPC layer, the surface of RC members was roughened purposely to create a good bond interface between UHPC and NSC substrate.

Two thicknesses, 25 mm and 50 mm, of UHPC layer were considered. Two slab specimens of each layer thickness were tested. One was not reinforced and the other had five 10 mm diameter high tensile steels (5T10) as longitudinal rebar as shown in Fig. 1. The five 12 mm diameter high tensile steels (5T12) (1.88% in ratio to the RC member section) was provided in both tension and compression to avoid macrocrack formation during the tests prior to the fracture of UHPC layer, and the additional 5T10 was installed to meet anchorage condition in UHPC layer. Table 1 shows the detailed geometry and area of rebar of the specimens.

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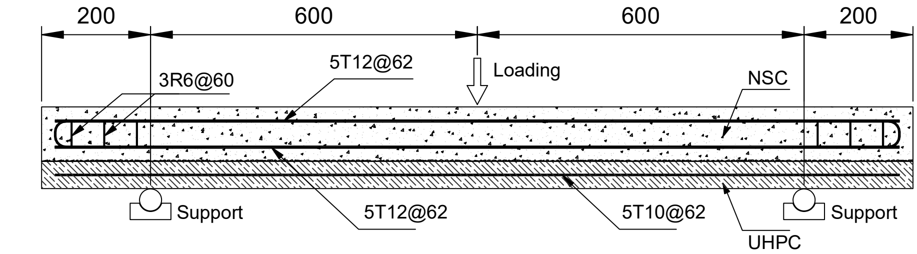
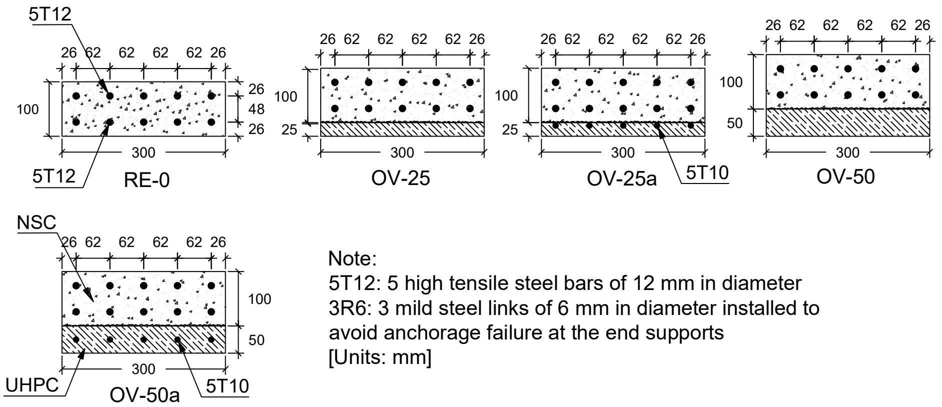


(a) Static test system (OV-50a)



(b) Cross-sectional layouts

Fig.1 Details of test setup and reinforcement arrangement [5]

Table 1 Details of the specimens [5]

Specimen	b_w (mm)	h (mm)	h_U (mm)	A_s (mm^2)	A'_s (mm^2)	A_{sU} (mm^2)
RE-0	300	100	-	565	565	-
OV-25	300	125	25	565	565	-
OV-25a	300	125	25	565	565	393
OV-50	300	150	50	565	565	-
OV-50a	300	150	50	565	565	393

Note: b_w = width of the specimens; h = total height of the specimens; h_U = thickness of the UHPC layer; A_s = area of the bottom rebar; A'_s = area of the top rebar; and A_{sU} = area of the rebar in UHPC layer

Table 2 Mechanical properties of concrete [5]

Material	Compressive strength (N/mm^2)	Flexural strength (N/mm^2)	Young's modulus§ (kN/mm^2)
NSC	23	-	22.5
UHPC	153	27.4	58.1

§: calculated using $E_c = 4700(f'_c)^{0.5}$ [7] (f'_c in MPa)

Table 3 Details of reinforcement properties [5]

Rebar	Yield strength (N/mm^2)	Young's modulus (kN/mm^2)
T12	502	200
T10	475	200

2.2 Material Properties

Mechanical properties of UHPC are listed in Table 2. UHPC with 3% volume steel fibres was adopted. Detailed UHPC mix design and preparation of the slabs could be found in [5]. Characteristics of the reinforcement are shown in Table 3.

2.3 Test Results

Fig. 2 shows load-deflection curves of the specimens. Owing to strengthening effect including increase of the specimen total height, the RC members with UHPC layer at the tension zone showed enhanced overall performance such as stiffness and load carrying capacity compared to the reference RE-0. In addition, the influence of reinforcing bars in UHPC did not seem to differ from those without rebar in initial stiffness as clearly seen in OV-50 and OV-50a. However, provided reinforcing bars in UHPC increased the ultimate load of the members.

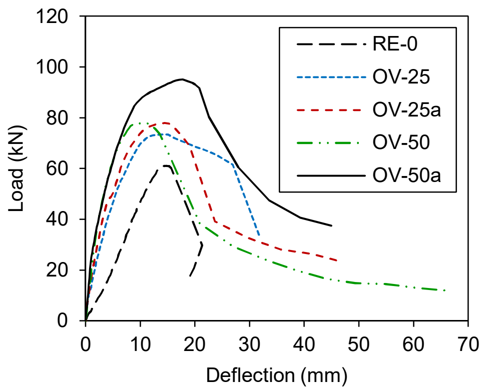


Fig. 2 Load-deflection curves [5]

Fig. 3 shows the crack patterns and failure mode of the specimens after test. The reference slab RE-0 showed brittle shear crushing and flexural cracks at the mid-span. The RC slabs strengthened with UHPC layer (OV-25, OV-25a, OV-50 and OV-50a) showed sudden shear cracks and debonding mode of UHPC layer. Further discussion on the experimental results could be found in [5].

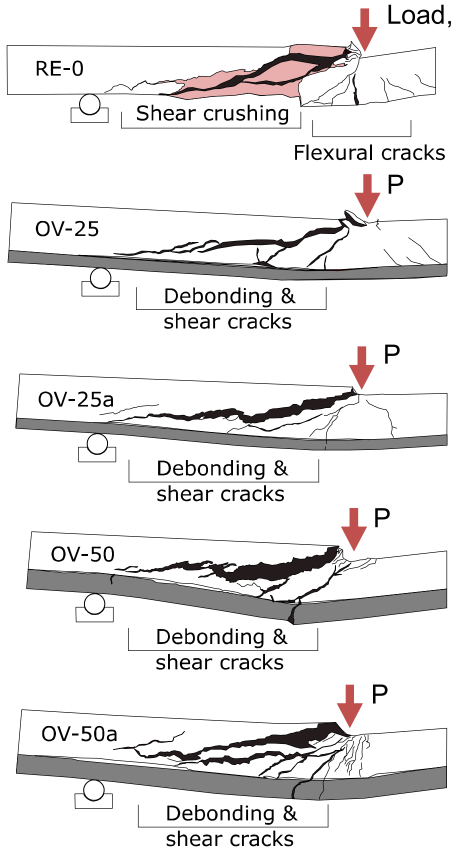


Fig. 3 Cracking behaviour of the specimens [5]

3. SUMMARY OF CURRENT DESIGN MODELS

3.1 Design Shear Models of RC Members

Current design provisions [7,9] for RC members suggest that the nominal shear capacity (V_n) can be calculated as a sum of the contribution of transverse steels (V_s) and the concrete (V_c) as follows:

$$V_n = V_s + V_c \tag{1}$$

For shear reinforcement (stirrups), V_s can be given as:

$$V_s = \frac{A_{sw} f_{yt} d}{s} \tag{2}$$

where A_{sw} is the area of shear reinforcement; f_{yt} is the yield strength of stirrups; d is the effective depth; and s is the spacing of stirrup.

a) ACI 318 Code

According to the ACI318 (2008) [7], the V_c can be expressed as:

$$V_c = \left(0.16\lambda\sqrt{f_c'} + 17\rho_s \frac{V_u d}{M_u}\right) b_w d \tag{3}$$

where λ is the reduction factor; f'_c (in MPa) is the compressive strength of concrete; ρ_s is the longitudinal ratio; V_u is the shear force; M_u is the moment at section; d is the effective depth; and b_w is the web width.

b) JSCE Recommendation

The JSCE recommendation (2007) [9] provides the expressions of V_c as follows:

$$V_c = \frac{\beta_d \beta_p \beta_n f_{vcd} b_w d}{\gamma_b} \tag{4}$$

where:

$$f_{vcd} = 0.2\sqrt[3]{f_{cd}'} \qquad \text{where } f_{vcd} \le 0.72 \text{ MPa}$$

$$\beta_d = \sqrt[4]{\frac{1000}{d}} \qquad \text{where } \beta_d \le 1.5$$

$$\beta_p = \sqrt[3]{100 p_v} \qquad \text{where } \beta_p \le 1.5$$

$$\beta_n = 1 + \frac{2M_0}{M_{ud}} \quad (N_d' \ge 0) \qquad \text{where } \beta_n \le 2$$

$$\beta_n = 1 + \frac{4M_0}{M_{ud}} \quad (N_d' < 0) \qquad \text{where } \beta_n \ge 0$$

where N'_d is the design axial compressive force; M_{ud} is the flexural capacity without consideration of axial force; M_0 is the flexural moment necessary to cancel stress due to axial force at extreme tension fibre; b_w is the web width; d (in mm) is the effective depth; p_v is the reinforcing bar ratio ($p_v = A_s/(b_w d)$); A_s is the area of tension reinforcement; f'_{cd} (in MPa) is the design compressive strength of concrete; and γ_b is the factor (suggested to be $\gamma_b = 1.3$).

3.2 Design Shear Models of FRC Members

The current design code, ACI544 [8], provides the nominal shear strength, V_n, of FRC members as follows:

$$V_n = \frac{2}{3} f_{ct} \left(\frac{d}{a}\right)^{0.25} b_w d \tag{5}$$

where f_{ct} is the tensile strength of FRC; a is the distance from the pointing load to the support; d is the effective depth; and b_w is the web width. It should be important to note that the empirical formulation does not account for factors that are recognised to significantly influence the shear strength, including the fibre length (or fibre types) and the longitudinal rebar ratio.

4. SHEAR CAPACITY OF RC MEMBERS STRENGTHENED WITH UHPC OVERLAY

4.1 Failure Mechanisms

To predict the shear capacity of RC members strengthened with UHPC overlay, investigation on failure mechanisms was carried out. UHPC layer strengthening to the soffit of RC members tends to undergo debonding mode in several different force transfer mechanisms as indicated in Fig. 4. Basically, the debonding occurs in the zone, where the shear and flexural moment are significant.

The debonding caused by shear generally occurs near the support that the inclined shear cracks induces the dowel action of the members spalling the concrete cover. The debonding crack then propagates along the bond interface of NSC and UHPC.

Flexural debonding behaviour of the members is the failure that either flexural cracks or inclined flexural-shear cracks at the mid-span are critically large. It then propagates to the support or the end of UHPC layer along the interface.

The shear debonding is often found to be the most critical behaviour due to the sudden and brittleness of the failure. Moreover, even the shear mechanism itself of RC members is complicated. The added UHPC layer provides more parameters, which should be considered into the shear models inevitably. Further studies based on particular tests such as bond/shear strength at the UHPC-NSC interface and the interaction of the dowel action and debonding are needed.

The interfacial bond behaviour remains a challenge in the development of accurate prediction methods. However, the following section described a simple method, which did not consider the effect of actual bond mechanical interface for simplicity as an initial study for predicting the shear capacity, based on modification of the existing design codes.

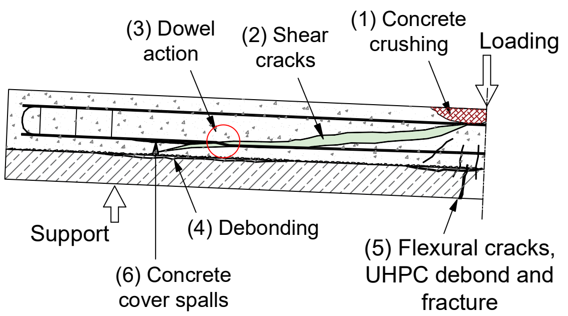


Fig. 4 Typical failure modes of RC members strengthened with UHPC

4.2 Method for Prediction of Shear Capacity

The nominal shear capacity of RC members strengthened with UHPC layer in this study was based on the existing design formulations [7-9]. Additionally, a suggestion from Noshiravani and Bruhwiler [1] and Yin et al. [5] that the shear capacity of the strengthened members are predominantly carried out by the web of the existing RC members was also taken into consideration. Based on their studies, it implies that the shear contribution of thin UHPC may be relatively small. The shear capacity relies on the tensile strength of UHPC, which may be determined by steel fibres. The effect of bond interface is not considered in Method (1) and (2) in this paper.

To predict the nominal shear resistance of RC members strengthened with UHPC, two different methods were individually proposed. Method (1) was investigated by converting the percentage of steel fibres (%Vol.) added in UHPC to the equivalent longitudinal steel ratio. Method (2) was adopted as a sum of two components ($V_c + V_U$), where V_c is the concrete shear strength of the RC members, and V_U is the contribution of UHPC. In the present calculation, for non-composite members, the effective depth d was defined as a distance from the extremely top compression fibre to the central gravity of bottom longitudinal rebar. For composite members consisted of tension UHPC layer, the d was given as a distance from extreme compression fibre to the centre of UHPC layer.

a) Method (1)

Method (1) was an extension of the current design codes for RC members, ACI318 [7] and JSCE [9] to the RC members strengthened with UHPC layer. The strengthened RC member was assumed as an equivalent RC member throughout the section by considering the contribution of steel fibres in UHPC to the equivalent longitudinal rebar. The equivalent steel ratio, ρ, was then given by:

$$\rho = \rho_s + \rho_{eq,F} \tag{6}$$

where ρ_s is the longitudinal steel ratio of the members and $\rho_{eq,F}$ is the equivalent ratio of the steel fibres.
The ρ_s is given as:

$$\rho_s = \frac{A_s}{b_w d} \tag{7}$$

where A_s is the area of longitudinal rebar in the members; d is the effective depth; and b_w is the width of the specimen sections.
The effective depth, d, was assumed as:

$$d = h_c + \frac{h_U}{2} \tag{8}$$

where h_c is the height of RC members; and h_U is the thickness of UHPC.
The $\rho_{eq,F}$ is given as:

$$\rho_{eq,F} = \%\mathrm{Vol.}\left(\frac{f_{ct}}{f_y}\right)\left(\frac{A_{UHPC}}{A_{RC}}\right) \tag{9}$$

where the tensile stress of UHPC $f_{ct} = 0.3(f'_c)^{2/3}$ was used where f'_c is the compressive strength of UHPC in MPa; f_y is the yield strength of longitudinal rebar; A_{UHPC} and A_{RC} are the area of UHPC and RC member, respectively, where $A_{UHPC} = b_w \times h_U$, and $A_{RC} = b_w \times d$. %Vol. is the percentage of steel fibres.

Therefore, expressions of the nominal shear strength of RC members strengthened with UHPC layer, $V_{n,OV}$, can be written as follows:

- Modification of ACI318

$$V_{n,OV} = \left(0.16\lambda\sqrt{f_c'} + 17\left(\rho_s + \rho_{eq,F}\right)\frac{V_u d}{M_u}\right) b_w d \tag{10}$$

where λ is the reduction factor; f'_c (in MPa) is the compressive strength of NSC; ρ_s is the longitudinal ratio; V_u is the shear force; M_u is the moment at section; d is the effective depth (Eq. (8)); and b_w is the web width.

- Modification of JSCE

$$V_{n,OV} = \frac{\beta_d \beta_p \beta_n f_{vcd} b_w d}{\gamma_b} \tag{11}$$

where:

$f_{vcd} = 0.2\sqrt[3]{f'_{cd}}$ where $f_{vcd} \leq 0.72$ MPa

$\beta_d = \sqrt[4]{\frac{1000}{d}}$ where $\beta_d \leq 1.5$

$\beta_p = \sqrt[3]{100(p_v + \rho_{eq,F})}$ where $\beta_p \leq 1.5$

$\beta_n = 1 + \frac{2M_0}{M_{ud}}$ $(N'_d \geq 0)$ where $\beta_n \leq 2$

$\beta_n = 1 + \frac{4M_0}{M_{ud}}$ $(N'_d < 0)$ where $\beta_n \geq 0$

where N'_d is the design axial compressive force; M_{ud} is the flexural capacity without consideration of axial force; M_0 is the flexural moment necessary to cancel stress due to axial force at extreme tension fibre; b_w is the web width; d (in mm) is the effective depth (Eq. (8)); p_v is the reinforcing bar ratio ($p_v = A_s/(b_w d)$); $\rho_{eq,F}$ is given by Eq. (9); A_s is the area of tension reinforcement; f'_{cd} (in MPa) is the design compressive strength of NSC; and γ_b is the factor (suggested value is $\gamma_b = 1.3$).

b) Method (2)

In this Method (2), each of the components, V_c and V_U, was assumed to contribute in shear resistance of the strengthened RC members, independently. The shear strength of the RC members, V_c, were calculated using the current design standards for the RC members. For the contribution of UHPC, V_U, the design guideline, ACI544 [8] was adopted.

The calculation in this method was based on a simple assumption that the RC members strengthened with UHPC layer was considered as two independent parts (NSC and UHPC) and simultaneously reached their ultimate shear strengths at the same time. Even though this assumption may not correspond to the shear patterns of the actual tests on UHPC layer of the slabs and the strain of the two materials at the maximum strength would be different, but for simplicity and for comparison with the other method, this assumption was adopted.

The formulations of nominal shear strength, $V_{n,OV}$, are then summarised as follows:

$$V_{n,OV} = V_{c,\text{ACI318}} + V_{U,\text{ACI544}} \tag{12}$$

where $V_{c,\text{ACI318}}$ was obtained by Eq. (3) for RC member. In Eq. (3), λ is the reduction factor; f'_c (in MPa) is the compressive strength of NSC; ρ_s is the longitudinal ratio; V_u is the shear force; M_u is the moment at section of the member; d is the effective depth (d = 74 mm); and b_w is the web width. For UHPC layer, $V_{U,\text{ACI544}}$ was given as:

$$V_{U,\text{ACI544}} = \frac{2}{3} f_{ct} \left(\frac{d}{a}\right)^{0.25} b_w d \tag{13}$$

where the tensile strength of UHPC, f_{ct}, was taken as $0.3(f'_c)^{2/3}$ where f'_c is the compressive strength of UHPC in MPa, and the effective depth of UHPC layer, d, was assumed as $h_U/2$. a is the distance from the pointing load to the support; and b_w is the web width; and h_U is the thickness of UHPC.

4.3 PREDICTION RESULTS AND VERIFICATION

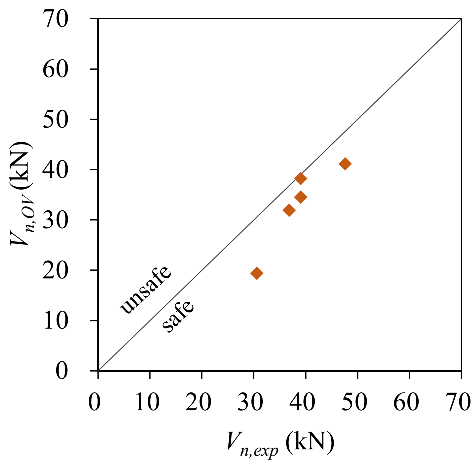
Table 4 shows the comparison between predicted and experimental results for the RC slabs strengthened with UHPC layer. The shear strength prediction was evaluated through investigation on the mean and coefficient of variation (COV) of $V_{n,exp}/V_{n,OV}$ ratios. $V_{n,exp}$ is the maximum shear force experimentally obtained as $V_{n,exp} = P_u/2$ where P_u is the ultimate load; and $V_{n,OV}$ is the predicted nominal shear force (Section 4.2).

The shear capacity predicted based on the modification of ACI318 (Eq. (10)) and JSCE (Eq. (11)) of Method (1) showed a good prediction with the mean $V_{n,exp}/V_{n,OV}$ ratio of 1.18 and 1.25, and COV of 18.8% and 14.6%, respectively. Meanwhile, Method (2) (Eq. (12)) gave a fair accuracy with the mean $V_{n,exp}/V_{n,OV}$ and COV of 1.31 and 15.9%, respectively.

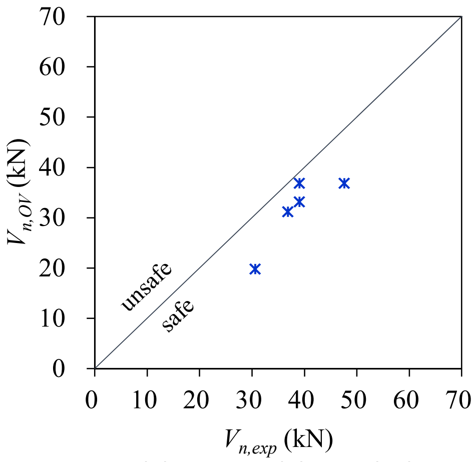
Fig. 5 shows the comparison between predicted nominal shear strength ($V_{n,OV}$) and test results ($V_{n,exp}$). From the figure, all design formulations provided good data points following the target line which represents $V_{n,OV} = V_{n,exp}$ and showing safe predictions as depicted below the target line.

Table 4 Comparison between predicted and test results

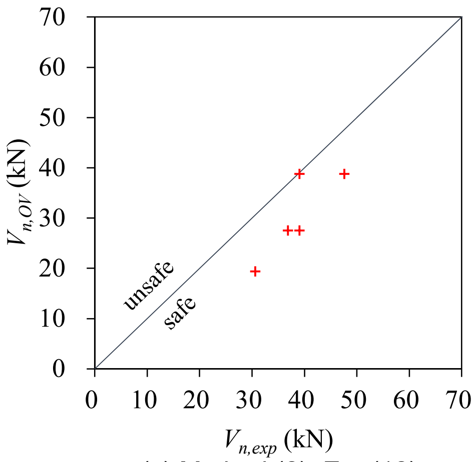
Experiment			Predicted shear strength					
			Method (1)		Method (1)		Method (2)	
			Eq. (10)		Eq. (11)		Eq. (12)	
Specimen	$V_{n,exp}$ (kN)	Failure mode	$V_{n,OV}$ (kN)	$V_{n,exp}/V_{n,OV}$ ratio	$V_{n,OV}$ (kN)	$V_{n,exp}/V_{n,OV}$ ratio	$V_{n,OV}$ (kN)	$V_{n,exp}/V_{n,OV}$ ratio
RE-0	30.54	Shear	19.45	1.57	19.83	1.54	19.45	1.57
OV-25	36.78	Shear	31.98	1.15	31.17	1.18	27.65	1.33
OV-25a	38.98	Shear	34.50	1.13	33.32	1.17	27.65	1.41
OV-50	38.99	Shear	38.23	1.02	36.78	1.06	38.60	1.01
OV-50a	47.53	Shear	41.33	1.15	36.84	1.29	38.64	1.23
Mean				**1.18**		**1.25**		**1.31**
COV				**18.8%**		**14.6%**		**15.9%**



(a) Method (1), Eq. (10)



(b) Method (1), Eq. (11)



(c) Method (2), Eq. (12)

Fig. 5 Comparison between predicted shear capacity and test results

5. CONCLUSIONS

Methods for predicting the shear capacity of RC members strengthened with UHPC overlay based on the existing design codes were presented. The shear capacity of the RC slabs strengthened with four UHPC configurations at the tensile zone were verified with experimental results.

From the assessments conducted in this study, the following conclusions could be made:

(1) The proposed modification of the existing design models for the strengthened slabs showed mean $V_{n,exp}/V_{n,OV}$ ratio and COV ranged from 1.18 to 1.31 and 14.6% to 18.8%, respectively.

(2) This study demonstrated a promising result in prediction of the shear strength for the UHPC layer strengthened RC slabs. However, further improvement considering the mechanical bond effect at UHPC-NSC interface should be conducted in the future.

ACKNOWLEDGEMENT

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