- Technical Paper -

FLEXURAL STRENGTH CALCULATION OF THE RC MEMBERS REHABILITATED WITH UHPC

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ABSTRACT

This paper addresses the calculation of flexural moment capacities of reinforced concrete (RC) members rehabilitated with ultra-high performance concrete (UHPC) at the tensile zone. The prediction was based on a simple modification of current design codes. To compute the flexural moment resistance in the composite section, rectangular stress block diagrams for compressive and tensile zone of the normal strength concrete member and UHPC layer were assumed, respectively. Results showed that the proposed method could predict the flexural moment compared to test results.

Keywords: ultra-high performance concrete, reinforced concrete, composite member, strengthening, repair, flexural moment

1. INTRODUCTION

Ultra-high performance concrete (UHPC) is a high strength and ductile material formulated by mixing reactive powder concrete with steel fibres. Recently, UHPC has been considered as a potential tool in the challenge of strengthening reinforced concrete (RC) structural elements. Concepts for using UHPC to strengthen parts of structures where the outstanding properties of UHPC could be fully exploited have been proposed by Brühwiler and Denarie [1]. To validate the concepts, four full-scale applications were discussed. It showed that UHPC development is suitable for use in either cast in-situ or precast applications.

Previous studies on the UHPC-concrete composite members showed that UHPC layer significantly enhances the structural performance such as the cracking development patterns, ultimate strength, and ductility [1-5] due to the excellent properties of UHPC showing strain hardening and energy absorption [6-11].

Although several studies on UHPC-concrete composite members have been experimentally conducted [2-5,12], few analytical models are available [2,13]. Alaee and Karihaloo [2] and Habel et al. [13] modelled UHPC-concrete members subjected to flexure. Moment-curvature relationships were computed through the cross-sectional analysis; however, several analytical steps were required.

For solely RC members, the flexural capacity can be calculated using current design codes. Current design codes for RC structures such as ACI318 [14] adopt a simplified stress block diagram for rectangular RC beams to compute the flexural moment. Whereas, the tensile strength of normal strength concrete (NSC) is generally negligible, that of UHPC should be taken into account since UHPC exhibits high tensile strength (> 8 MPa). It is evidently shown in current design guidelines of fibre reinforced concrete (FRC) members such as ACI544 [15].

To date, design provisions have not yet been available for UHPC-concrete composite members. Methods that can predict its structural capacity are therefore needed. Use with modification of the existing design models of RC and/or FRC structures would be useful because it is simple and easy-to-use. The analytical models should be derived from the principle concepts of the current design guidelines for RC and/or FRC structures.

The objective of this paper is to introduce a method for calculating the flexural moment capacity of RC members rehabilitated with UHPC through a simple modification of the existing models of the current design codes.

In this paper, five UHPC-concrete composite slabs tested by Yin et al. [5] are selected and used to validate the proposed method. A summary of the current design models [14,15] is provided. Simple assumptions for modification of the current design formulations adopted in this study are described. Comparison between the prediction using proposed modification of the existing design codes and the test results is presented.

2. BRIEF DESCRIPTION OF THE SPECIMENS AND TEST RESULTS

2.1 Geometry of the Specimens

This section briefly describes the experimental specimens and the test results of RC slabs strengthened

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with various UHPC thicknesses, which were used to validate the analytical models in this study. Full details of the experiment could be found in [5].

Each specimen was 1600 mm long with a clear span of 1200 mm and 300 mm × 100 mm cross-section. Different UHPC thicknesses were applied in the tension zone as patch material for repair and rehabilitation of structural members. Often in practice, deteriorated concrete is removed and repair materials are applied to the concrete substrate. The different thicknesses of the specimens were used to reflect extents of deterioration and repair.

As shown in Fig. 1, all the slab specimens used in this study (RE-series in [5]) had five 12 mm diameter high tensile steel bars (5T12) at the top and bottom. Table 1 shows the geometry and area of reinforcement of the slab specimens. In Table 1, b_w is the width of the specimen; h is the total height of the specimen; h_U is the thickness of the UHPC layer; A_s is the area of the bottom longitudinal rebar; and A'_s is the area of the top rebar.

2.2 Material Properties

Mechanical properties of NSC and UHPC are listed in Table 2. For UHPC, 3% volume steel fibres were adopted. Straight steel fibres of 13 mm in length and 0.2 mm in diameter were used. Detailed UHPC mix design and preparation of the specimens could be found in [5]. Characteristics of the reinforcement are shown in Table 3.

Table 1 Geometry and area of reinforcement of the slab specimens (RE-series in [5])

Specimen	b_w (mm)	h (mm)	h_U (mm)	A_s (mm^2)	A'_s (mm^2)
RE-0	300	100	-	565	565
RE-20	300	100	20	565	565
RE-32	300	100	32	565	565
RE-50	300	100	50	565	565
RE-100	300	100	100	565	565

Table 2 Concrete properties of the specimens [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 $E_c = 4700(f'_c)^{0.5}$ [14] (f'_c in MPa)

Table 3 Reinforcing rebar properties of the specimens [5]

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

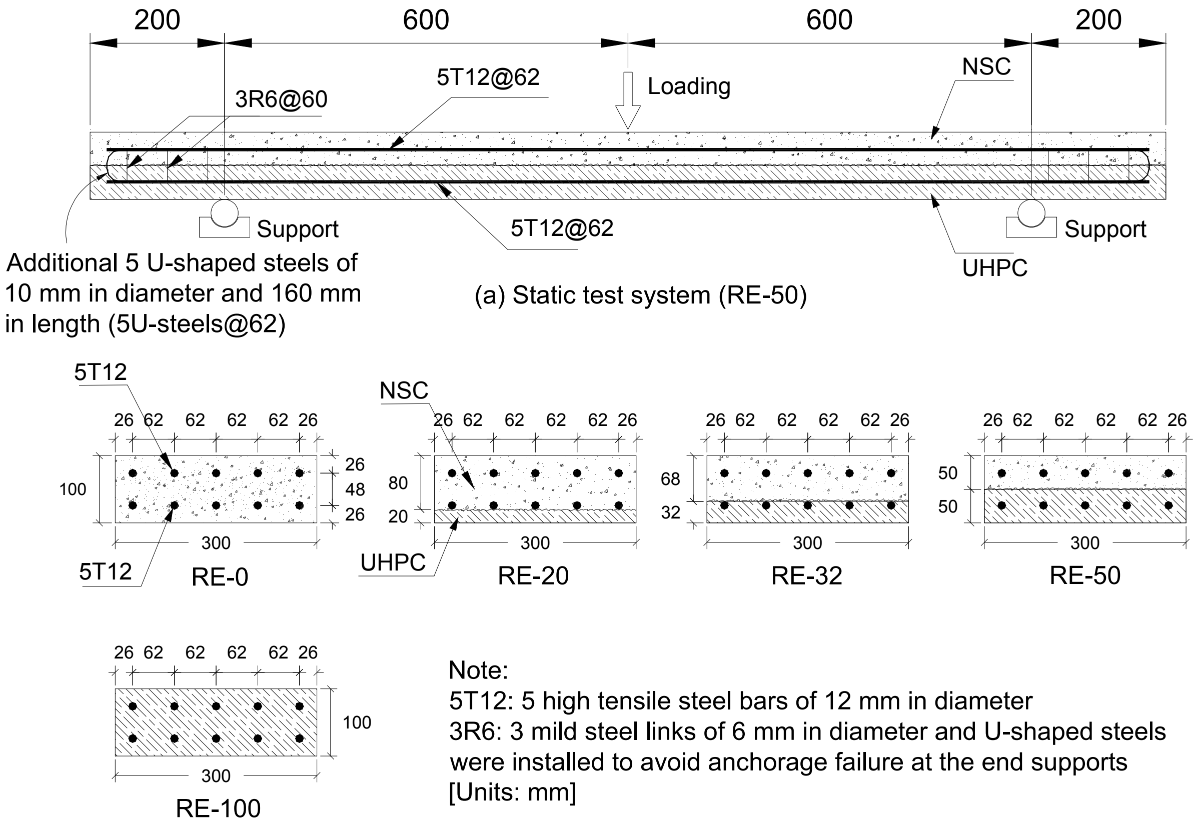

Fig.1 Static test system and cross-sectional layouts of the specimens [5]

2.3 Test Results

Fig. 2 shows the illustration of failure cracking patterns developed in the specimens. Slab RE-0 made of NSC failed in shear with concrete crushing along the shear span. Whereas slab RE-20 showed some shear cracks and debonding, slabs RE-32 and RE-50 exhibited flexural failure with concrete crushing at the compression zone and there were no visible signs of debonding of the UHPC layer.

Load-deflection curves of the specimens are shown in Fig. 3. As can be seen in Fig. 3, all rehabilitated slabs (i.e., RE-20, RE-32, and RE-50) exhibited extensive deflection hardening and ductility during the post cracking range. Although no strength improvement was found in any of the rehabilitated slabs as compared to RE-0, it could be offset by their excellent energy absorption capabilities. RE-32 failed at a lower peak load because UHPC layer that reached up to the top of reinforcement prevented effective shear transfer between longitudinal reinforcement and concrete. It was reported that the bottom longitudinal reinforcement yielded in all the specimens except RE-0. Further details of discussion and test results could be found in [5].

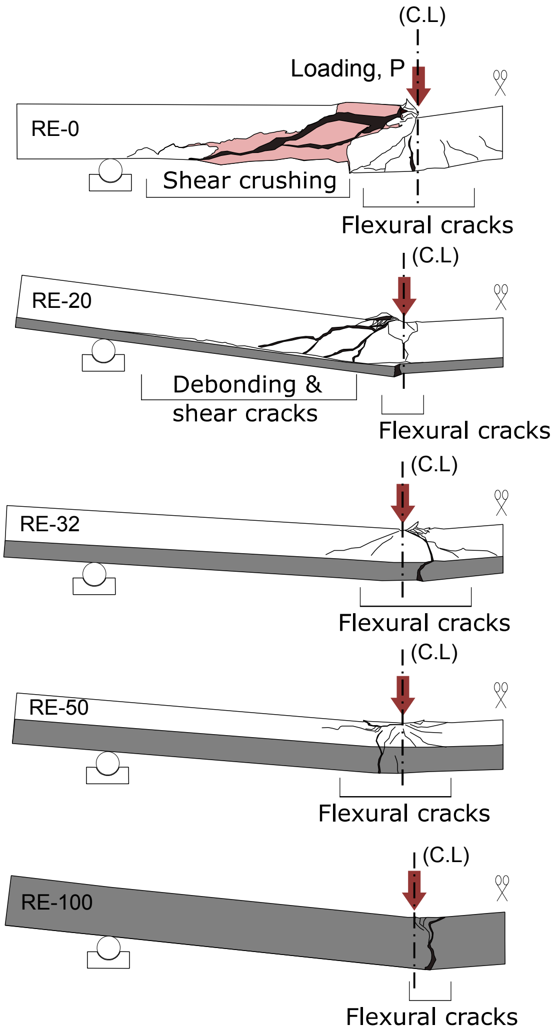

Fig. 2 Cracking behaviors of the specimens after the static loading test [5]

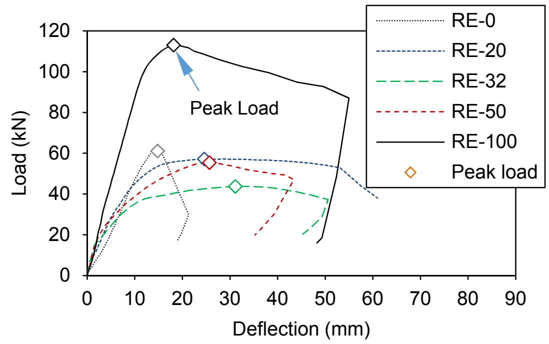

Fig. 3 Load-deflection curves of the specimens obtained from the test [5]

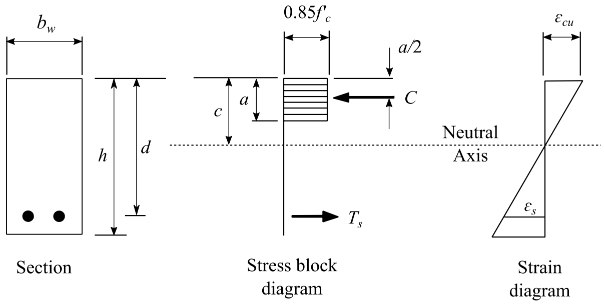

Fig. 4 Existing design assumptions of flexural moment for RC member [14]

3. EXISTING FLEXURAL DESIGN MODELS

3.1 ACI 318 Code for RC Members

The current design code for RC members, ACI318 [14] suggests that the nominal moment resistance, M_n, can be calculated based on simplified rectangular stress block diagram and the tensile stress of concrete is neglected as shown in Fig. 4. The expression of M_n is given as follows:

$$M_n = A_s f_y \left(d - \frac{a}{2} \right) \tag{1}$$

where A_s is the area of reinforcing bar; f_y is the yield strength of reinforcing bar; d is the effective depth; and a is the depth of rectangular stress block.

3.2 ACI 544 Code for FRC Members

Current design provisions for FRC members such as ACI544 [15] are based on the principle of the stress-strain relationship for computing the flexural moment resistance.

The basic design assumptions of ACI544 are shown in Fig. 5. The calculated value of nominal moment resistance for FRC members, M_n, can be achieved by equilibrium equations through the sectional analysis as follows:

$$M_n = A_s f_y \left(d - \frac{a}{2} \right) + \sigma_t b_w (h - e) \left(\frac{h}{2} + \frac{e}{2} - \frac{a}{2} \right) \tag{2}$$

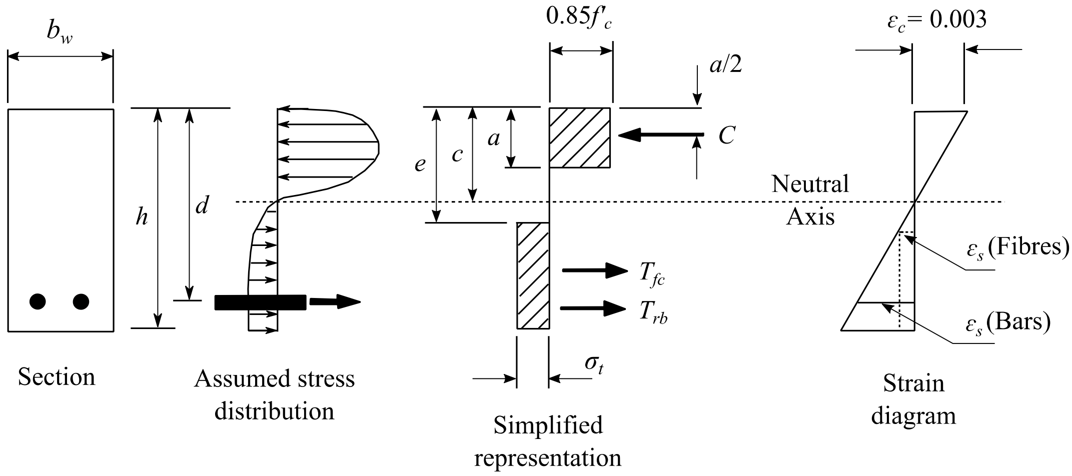

Fig. 5 Existing design assumptions of flexural moment for FRC member [15]

where A_s is the area of reinforcing bar; f_y is the yield strength of reinforcing bar; b_w is the web width; a is the depth of rectangular stress block; h is the height; d is the effective depth; σ_t is the tensile stress in fibrous concrete; and e is the distance from extreme compression fibre to top of tensile stress block of fibrous concrete.

The e is given as follows:

$$e = [\varepsilon_s\,(\text{fibres}) + 0.003]\left(\frac{c}{0.003}\right) \tag{3}$$

where ε_s(fibre) is the tensile strain in fibres; and c is the neutral axis depth;

The σ_t (in MPa) can be calculated as follows:

$$\sigma_t = 0.00772\left(\frac{l}{d}\right)\rho_f F_{be} \tag{4}$$

where l is the fibre length; d is the effective depth; ρ_f is the percent by volume of steel fibres; and F_{be} is the factor of bond efficiency of the fibre [15]. In this paper, $F_{be} = 1.0$ was used in the calculation for each specimen.

4. FLEXURAL STRENGTH OF THE RC MEMBERS REHABILITATED WITH UHPC

4.1 Theory

The experimental results of the RC members rehabilitated with UHPC used in this study mainly failed into two modes: fracture of UHPC and concrete crushing. The flexural strength of the members could be predicted using the equilibrium with geometrical compatibility through a section of the members. For fracture of UHPC, the compressive stress of concrete (NSC) may not reach its maximum strength. Then, the stress block diagram should be modified accordingly to the stress level. Experiments particularly for this issue are necessary to be conducted. In this study, however, the concrete strain of $\varepsilon_c = 0.003$ was used since no experiments were performed. In addition, both the fracture of UHPC and concrete crushing were assumed to fail simultaneously.

An assumed representation of stresses in the UHPC-concrete composite section is depicted in Fig. 6. The tensile stress, σ_t, used for FRC members in Fig. 5, was extended for UHPC layer of the UHPC-concrete composite members. Since the UHPC layer at the tension chord is relatively thin thickness, it may be reasonable to assume that the distance, $h - e$, (Fig. 5) shall be taken as the UHPC thickness, h_U (= $h - e$) as depicted in Fig. 6, where h is the total height of the section. A tensile force at the UHPC layer, T_{UHPC}, was then computed as the production of the stress, σ_t, which for instance was obtained from Eq. (4), and the corresponding area, A_{UHPC} (= $b_w \times h_U$).

4.2 Equilibrium Condition

Similarly to the design codes, an equilibrium equation was derived from the compatibility condition, which the strain varies linearly along the cross-section as shown in Fig. 6. In equilibrium condition, the equation could be expressed as:

$$C_c + C_{sc} = T_{st} + T_{UHPC} \tag{5}$$

where:

$$C_c = 0.85 f'_c\, \alpha x_n b_w$$
$$C_{sc} = A'_s \sigma'_s$$
$$T_{st} = A_s \sigma_s$$
$$T_{UHPC} = A_{UHPC}\sigma_t$$

The flexural moment capacity, M_{fle}, is then given by:

$$M_{fle} = A_s\sigma_s\left(d - \frac{\alpha x_n}{2}\right) + A_{UHPC}\sigma_t\left(d_U - \frac{\alpha x_n}{2}\right) + A'_s\sigma'_s\left(\frac{\alpha x_n}{2} - d'\right) \tag{6}$$

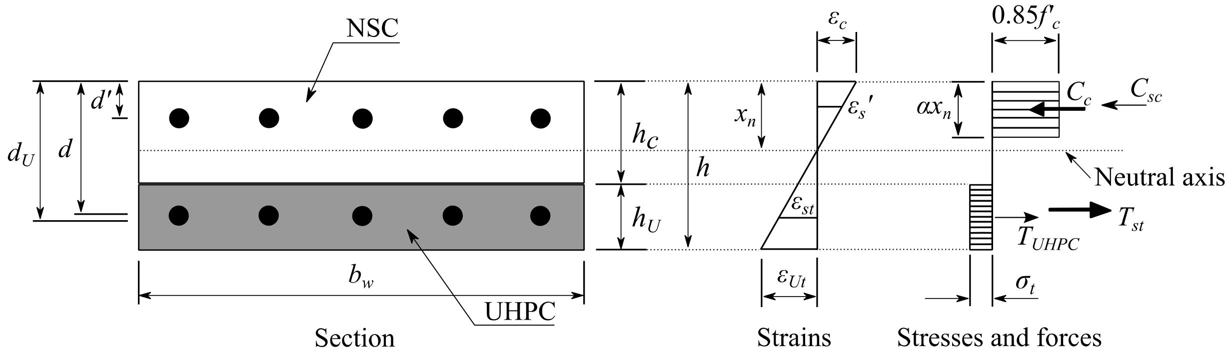

Fig. 6 Schematic representatives of stresses and strains in a section for RC member rehabilitated with UHPC

Note:

b_w = web width.
d = effective depth.
d' = depth from the extremely top surface to the center of top reinforcement.
d_U = depth from the extremely top surface to the center of UHPC layer ($d_U = h_C + h_U/2$).
h_C = depth of NSC.
h_U = thickness of UHPC layer.
x_n = distance between extremely top surface and neutral axis in a section.
f'_c = compressive strength of concrete.
α = factor relating depth of equivalent rectangular compressive stress block to neutral axis depth and it shall be taken as 0.85 in this study due to f'_c = 23 MPa according to [14].
A'_s = area of the top reinforcement.
A_s = area of the bottom reinforcement.
A_{UHPC} = area of UHPC ($A_{UHPC} = b_w \times h_U$).
σ'_s = stress of the top reinforcement.
σ_s = stress of the bottom reinforcement.
σ_t = tensile stress of UHPC obtained using Eq. (4).

In the analysis, the situations can be classified as follows:

- No reinforcing bar yields ($\sigma'_s < f_y$ and $\sigma_s < f_y$), where f_y is the yielding strength of rebar.
- Tension reinforcement (bottom rebar) yields and compression reinforcement (top rebar) does not yield ($\sigma_s = f_y$ and $\sigma'_s < f_y$).
- Both tension and compression reinforcement yield ($\sigma_s = f_y$ and $\sigma'_s = f_y$).

The distance to the neutral axis, x_n, can be calculated using strain compatibility and equilibrium condition, and checking the strain level in the reinforcement rebar. In this study, Eq. (6) was used to calculate the flexural moment for all the specimens. It should be mentioned that for RE-0, the second term of Eq. (6) became zero due to no UHPC contribution. For RE-100, the distance, $h - e$, shown in Fig. 5 equal to 50 mm and d_U in Eq. (6) equal to 75 mm were assumed.

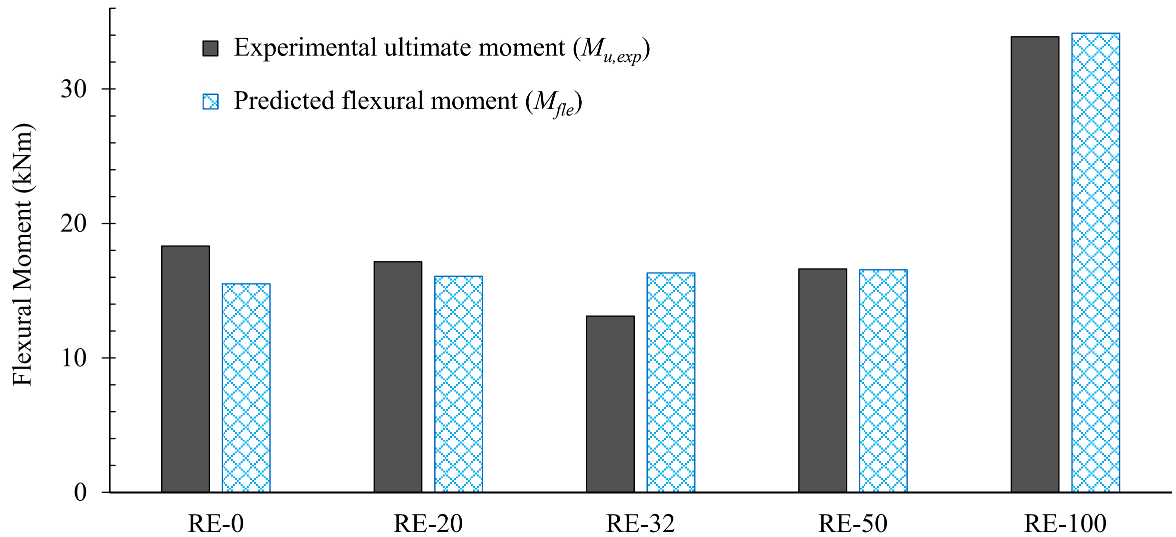

Fig. 7 Comparison between the prediction and experimental results of the flexural moment capacities of the specimens

5. PREDICTION RESULTS

Fig. 7 shows comparison between the prediction and experimental results for the RC members rehabilitated with UHPC. In Fig. 7, the ultimate moment, $M_{u,exp}$, was experimentally obtained as $M_{u,exp} = V_{n,exp} \times a$, where a is the distance from loading to support; the shear force, $V_{n,exp}$, was experimentally obtained as $V_{n,exp} = P_u/2$, where P_u is the ultimate load; and M_{fle} is the predicted flexural moment. From the results, it showed that the prediction (M_{fle}) agreed well with the experimental ultimate moment ($M_{u,exp}$).

As the thickness of UHPC layer increased, the calculated moment slightly increased except RE-100. This result indicated that the contribution of tension fracture of UHPC layer was not significant for the specimens used in this study.

The specimen RE-100 attained extremely high moment capacity for both the experimental and calculation results among all the specimens. The main reason to this was that only RE-100 had UHPC in the compression zone and the high compressive strength of UHPC affected the increase of flexural capacity.

In the calculation, bottom longitudinal rebar yielded for each specimen. The results from the calculation roughly agreed with the response of the experiments, which showed yielding of longitudinal reinforcement for all the specimens except RE-0.

6. CONCLUSIONS

The flexural strength calculation of the RC members rehabilitated with UHPC based on the existing design codes was presented. From the assessments conducted in this study, the following conclusions could be made.

Using the adopted method, the flexural moment capacities of the five slab specimens were obtained and compared to the experimental results. It showed that the ultimate flexural moments can be predicted with good accuracy.

Although the prediction of the flexural moment capacities in this study performed well, further study should be carried out to develop and improve the model for concrete structures rehabilitated with UHPC.

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